United States
Department of
Agriculture

Soil
Conservation
Service

National Engineering Handbook

Section 4

Hydrology

NATIONAL ENGINEERING HANDBOOK

SECTION 4

HYDROLOGY

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NATIONAL ENGINEERING HANDBOOK

SECTION 4

HYDROLOGY

CHAPTER 1. INTRODUCTION

by

Victor Mockus
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1964

Reprinted with minor revisions, 1969

SCS NATIONAL ENGINEERING HANDBOOK

SECTION 4

HYDROLOGY

CHAPTER 1--INTRODUCTION

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SOIL CONSERVATION SERVICE

National Engineering Handbook

Section 4

HYDROLOGY

CHAPTER 1. INTRODUCTION

The SCS National Engineering Handbook (NEH) is intended primarily for Soil Conservation Service (SCS) engineers and technicians. It presents material needed to carry out SCS responsibilities in soil and water conservation and flood prevention. Section 4, HYDROLOGY, contains methods and examples for studying the hydrology of watersheds, for solving special hydrologic problems that arise in planning watershed-protection and flood-prevention projects, for preparing working tools needed to plan or design structures for water use, control, and disposal, and for training personnel newly assigned to activities that include hydrologic studies.

SCOPE. Section 4 contains some new techniques that were developed by SCS personnel to meet specific needs of SCS. Well-known techniques from other sources are included where necessary to illustrate special applications to watershed-project planning, evaluation, and design. Hydrologic theory is held to the minimum necessary to show the development of methods not readily available elsewhere. References to hydrologic literature are given if they provide additional theory, data, discussion, or details of a method.

Each major kind of hydrologic problem is discussed, and where possible, alternative solutions are given and their relative merits are briefly considered. Descriptive material is kept to a minimum. All equations and examples are numbered for ease of reference. The

section is so arranged that each principal subject is discussed in a separate chapter, and cross-references to other chapters are made as needed. The table of contents is a reference to specific topics, methods, and examples; the glossary (chap. 22) is a reference to specific terms.

DUTIES AND RESPONSIBILITIES OF SCS HYDROLOGISTS

Memorandums from the director of the SCS Engineering Division define the technical duties and responsibilities of SCS hydrologists. Among the more important responsibilities is that of choosing the most suitable hydrologic method to use for a given problem. SCS engineering projects that require some application of hydrology may range in construction cost from a few hundred dollars to several million dollars. A hydrologic method suitable at one end of this range is usually unsuitable at the other. Two projects of about the same cost may require widely different methods because of differences in available data or location of benefits or topography. The chosen method in each case must be adequate to arrive at sound conclusions in terms of conditions, objectives, and functions of the project. The advice of the Engineering and Watershed Planning Unit hydrologist should be sought if there is doubt about the suitability of a method. For studies in which the choice of method is limited by available survey time or funds, the results must be regarded as tentative pending an investigation of sufficient scope.

Because the watershed work plan party works as a team, the hydrologist must be familiar with the work and needs of the economist, geologist, design engineer, and others who will use the results of a hydrologic study. To familiarize others with his own work and needs, the hydrologist must be able to describe the theories and working details of his methods, to say what data are required, what calculations are made and how they are made, and to give the approximate number of man-days of work needed to complete a job.

<u>OTHER TECHNICAL GUIDES</u>. SCS hydrologists should have and be familiar with other national guides and handbooks used in SCS. Publications of special interest are:

1. Watershed Protection Handbook
2. Economic Guide for Watershed Protection and Flood Prevention
3. SCS National Engineering Handbook:

Section 5 - Hydraulics

Section 15 - Irrigation

Section 16 - Drainage

4. Technical releases

5. Handbooks issued by State offices of SCS.

It is also necessary to be familiar with handbooks, manuals, and other in-service publications of the other agencies in a cooperative study. It may be necessary to use both SCS methods and those of a cooperating agency in order to meet, as nearly as possible, the requirements of both agencies. But SCS methods must be used for SCS projects unless approval to use other methods is obtained from the director of the SCS Engineering Division.

SCS hydrologists are expected to keep up-to-date on new developments in hydrology by reading technical papers in transactions, proceedings, or journals of organizations such as the American Society of Agricultural Engineers, American Society of Civil Engineers, Society of American Foresters, American Geophysical Union, Soil Conservation Society of America, and Soil Science Society of America. The solution of hydrologic problems requires a knowledge of several interrelated sciences, and hydrologists must accept every opportunity to increase their knowledge of the geology, soils, plant life, climatic variations, and agricultural practices of their assigned areas.

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SECTION 4

HYDROLOGY

CHAPTER 2. PROCEDURES

by

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CHAPTER 2--PROCEDURES

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CHAPTER 2. PROCEDURES

Hydrology for the evaluation of watershed-protection and flood-prevention projects is one of the major concerns of this handbook. The evaluation is a detailed investigation of present (no project) and future (with project) conditions of a watershed to determine whether given objectives will be met, and it is the basis on which recommendations for or against the project are founded. A summary of the evaluation is included in a work plan, which is the official document for carrying out, maintaining, and operating the project. The hydrology is not difficult, but it is complex. The procedures discussed in this chapter serve both as a guide to the hydrology and as a unifying introduction to succeeding chapters of Section 4, Part I.

A project evaluation begins with a preliminary investigation (PI), which is a brief study of a potential project in order to estimate whether a detailed investigation is justified (chap. 3). If it is, information from the PI is used in writing a work outline that gives the desired scope, intensity, and schedule of the planning study, its estimated cost, the personnel needed, and the completion date for a work plan. An important part of the planning study is the hydrologic evaluation, in which data collection, computation, and analysis are equally important divisions of work. Availability governs the collection of data. Size or cost of project influences the choice of computational and analytical methods (chap. 1). SCS policy determines the number and kind of analyses. Nevertheless the basic procedure of the evaluation does not vary. It is flexible, since some tasks can be done simultaneously or in a preferred sequence and nearly all tasks can be done by a preferred method, but the general plan is invariable. The work outline schedule follows the plan in principal. The plan, schedule, and chapters in Section 4, Part I, are related as follows:

Data collection. Base maps (chap. 3) and rainfall (chap. 4) and runoff (chap. 5) data are collected early in the study. Field surveys provide stream cross sections and profiles (chap. 6) and damsite maps. Interviews with local SCS personnel provide data on hydrologic soil-cover complexes (chaps. 7, 8, and 9).

Computations. Storm runoffs (chap. 10), snowmelt runoffs (chap. 11), special effects of land use and treatment (chap. 12), and the relations of stream stages to inundation (chap. 13) and discharge (chap. 14) are computed early in this phase of the study. Travel times

and lags (chap. 15) are computed for use in hydrograph construction (chap. 16) and flood routing (chap. 17). Runoff or peak discharge frequencies (chap. 18), transmission losses (chap. 19), and watershed yield (chap. 20) are computed only if they are required in the study.

Analyses. Three conditions of a watershed are studied in accordance with SCS policy. In order of study they are:

1. Present. Condition of the watershed at the time of the survey; and the base to which the proposed project is added.

2. With future land use and treatment (LU&T) measures. Proposed LU&T measures are added to the first condition. The measures are described in the Watershed Protection Handbook.

3. With future LU&T measures and structures. Watershed-protection and flood-prevention structures are added to the second condition. The structures are described in the Watershed Protection Handbook.

This order makes the analysis fall into a natural sequence in which measures that are first to affect runoff are first to be evaluated. Flood routings for the present condition give the discharges from which present flood damages are computed in the economic evaluation. The routings are modified (chap. 12) to give discharges for determining the effects of LU&T. New routings or further modifications (chap. 17) are made for the third condition to give discharges for determining the effects of structures. Generally it is the third condition that is studied at great length because an optimum number and location of structures is desired. Final design of individual structures is made late in the investigation or after the work plan is approved. The hydrology and SCS hydrologic criteria for design are given in chapter 21.

ELECTRONIC COMPUTER PROGRAMS. Work in the computational phase is reduced by use of an electronic computer program for determining water-surface profiles from which stage-inundation and stage-discharge relations are obtained. * In both the computational and analytical phases * work is reduced by use of a program that computes runoff, constructs hydrographs, routes hydrographs through reservoirs and stream channels, and combines routed or unrouted hydrographs. The print-out consists of stages, peak discharges, and detailed hydrographs, as desired, for natural or design storms and for any combination of structures. This program is described in SCS Technical Release 20.

FLOW CHARTS

The sequence of work in the hydrologic evaluation is shown in figure 2.1. The forms of maps, graphs, and tables are simplified representations of the various standard forms used in the different States. The PI, which precedes the evaluation, is discussed in chapter 3. The design hydrology, which comes latest, is shown in figure 2.3; details are given in chapter 21.

After evaluation for the first condition is completed it is not necessary to repeat some of the early steps for the remaining evaluations. The second evaluation starts with hydrologic soil-cover complexes, the third with unit hydrographs for "with structures", then the evaluations proceed in the same way as the first except for obvious omissions.

Of the basic data needed in the evaluation only the historic rainfall and streamflow data are likely to be unavailable; the rest are obtainable from field surveys. Lacking rainfall and runoff data, the procedure goes as shown in figure 2.2. The rainfall-frequency data shown in the figure are from U.S. Weather Bureau publications (chap. 4). Direct checks on runoff cannot be made, but indirect checks can be made if nearby watersheds are gaged (see table 5.2).

Some steps in the procedures of figures 2.1 and 2.2 are taken in an entirely different way in the two methods for regional analysis and concordant flow.

REGIONAL ANALYSIS METHOD. This method is for estimating the magnitudes and frequencies of peak discharges or runoff volumes for ungaged watersheds by use of relationships for nearby gaged watersheds. Some of the hydraulic work, construction of hydrographs, and flood routing are reduced or eliminated from the evaluation but not from the design hydrology. The method in its simplest form is as follows:

1. Select nearby gaged watersheds that are climatically and physically similar to the ungaged watershed. These watersheds and nearby areas like them comprise the region that gives the method its name.

2. Construct frequency lines (chap. 18) for peak discharges or runoff volumes of the gaged watersheds.

3. Plot peak discharges or runoff volumes for selected frequencies (only the 2- and 100-year frequencies if the frequency lines are straight) of each gaged watershed against its drainage area size, using log paper for the plotting and making straight-line relationships for each frequency.

4. Construct the frequency line for the ungaged watershed (or any of its subdivisions) by entering the plot with drainage area, finding the magnitudes at each line of relationship, plotting the magnitudes at their proper places on probability paper, and drawing the frequency line through the points.

5. Apply the frequency lines of step 4 in the procedure for present conditions. Discharges or volumes for with-project conditions are obtained by use of auxiliary relationships described in chapters 12 and 17.

In practice the method is more complex but usually only in step 3: variables in addition to drainage area are related to the peaks or volumes. The variables are one or more of the following, alone or in combinations, directly or by means of index numbers: type of climate, mean annual precipitation or rainfall or snowfall, mean seasonal precipitation or rainfall or snowfall, maximum or minimum average monthly rainfall, storm pattern, storm direction, x-year frequency y-hour duration rainfall, mean number of days with rainfall greater than x inches, mean annual number of thunderstorm days, mean annual or seasonal or monthly temperature, maximum or minimum average monthly temperature, orographic effects, aspect, stream density, stream pattern, length of watershed, length to "center of gravity" of watershed, length of main channel, average watershed width, altitude, watershed rise, main channel slope, land slope, depth or top width of main channel near outlet for x-year frequency discharge, time of concentration, lag, time to peak, percentage of area in lakes or ponds, extent or depth of shallow soils, extent of major cover, hydrologic soil-cover complex, geologic region, infiltration rate, mean base flow, mean annual runoff, and still others. Combinations of these variables are used as single variables in the analysis, one such combination being the product of watershed length and length to center of gravity divided by the square root of the main channel slope. Index numbers (chap. 18) are used for variables (such as geologic region) not ordinarily defined by numerical values.

The use of multiple regression methods (chap. 18) is a necessity if more than one variable appears in the relationship. There is only one adequate measure of the accuracy of the relationship (therefore of the regional analysis) and this is the standard error of estimate in arithmetic units. Computation of the error is illustrated in chapter 18.

<u>CONCORDANT FLOW METHOD</u>. This method can be applied only if storm rainfall and high-water mark (HWM) data are available for a large general storm and flood over the watershed. In States where the method is regularly used the data are obtained after such a flood on any watershed with a potential for a project and stored until needed. When the project evaluation is to be made the stored data are supplemented by data from the usual field surveys (chap. 6).

In the concordant flow method the isohyetal map (chap. 4) for the storm producing the large general flood supplies the average rainfall depth for the drainage area above the cross section at each HWM. The average hydrologic soil-cover complex (chaps. 9 and 10) above each section and the rainfall give the direct runoff (chap. 10). The flood discharge at each HWM is computed (chap. 14), reduced to the discharge for one inch of runoff, and plotted on log paper against the drainage area. The slope of a straight line fitting the plotting gives the exponent used later in the concordant flow routing equation (chap. 17). It is this plotting that gives the method its name (the flows on the line are "concordant"). The Manning's n (chap. 14) for a discharge plotting far from the line is adjusted to make the point fall more nearly on the line. The adjusted n is used in computing the stage-discharge curve (chap. 14) at the section.

Runoffs for a historical series of storms are used with the discharge-area plotting to give peak discharges for the present condition. Runoffs are modified (chap. 12) to give the effects of LU&T and get the discharges for the second condition. The concordant flow equation (chap. 17) is used to get discharges for the third condition.

Limitations on the method are those that apply to any method using watershed averages: storm rainfalls must be approximately uniform over the drainage areas, structural measures must be uniformly distributed (chap. 17), and effects of channel improvements must be minor in comparison with effects from structures.

Design Hydrology

The storages and spillway capacities of floodwater-retarding structures are determined as shown by the flow chart in figure 2.3. Chapter 21 gives details of the various steps and provides the SCS criteria of the design hydrology. That chapter also contains design hydrology in outline form for channel improvement, levees, and minor project or on-farm structures.

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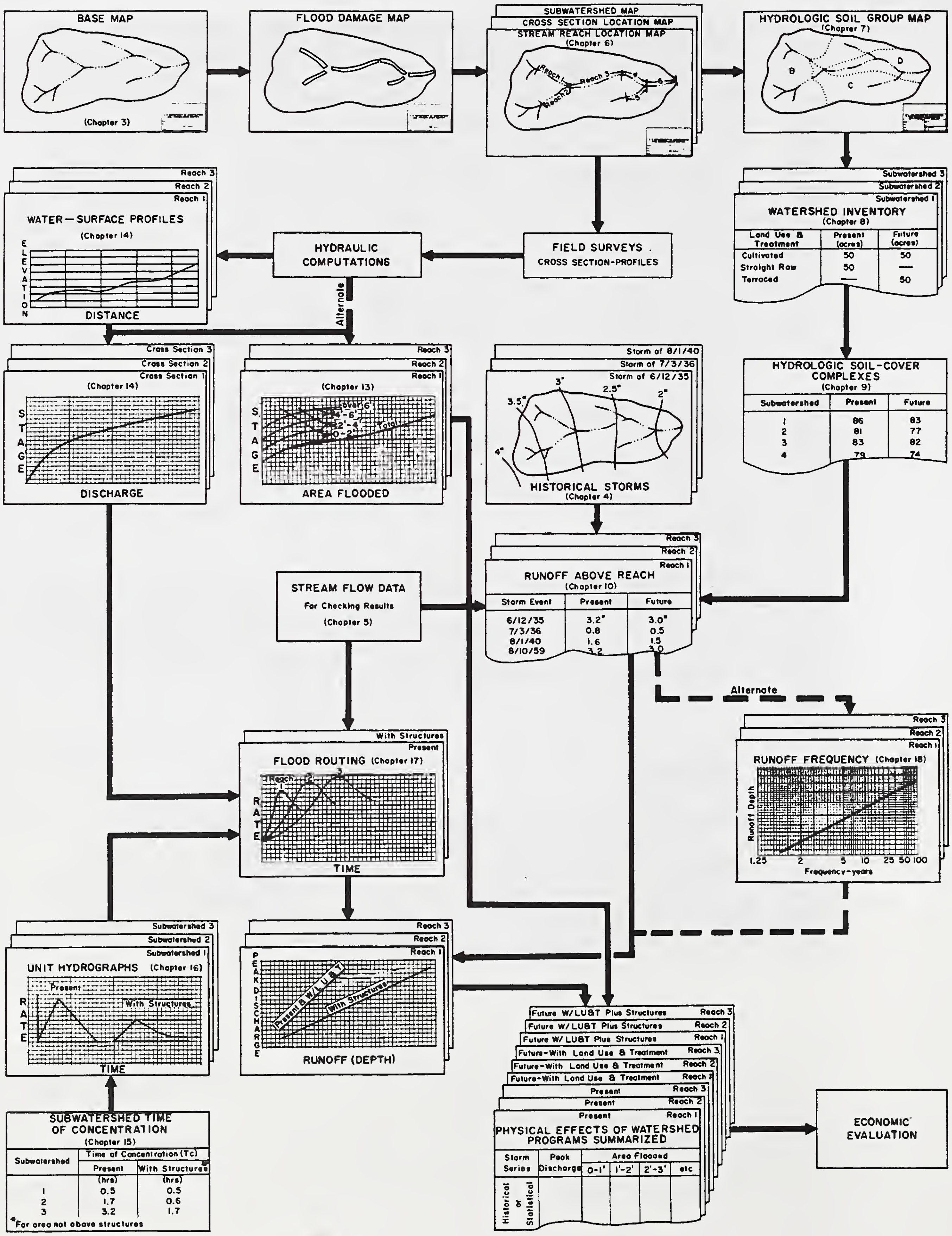

FIGURE 2.1-General process hydrology of watershed project evaluation.

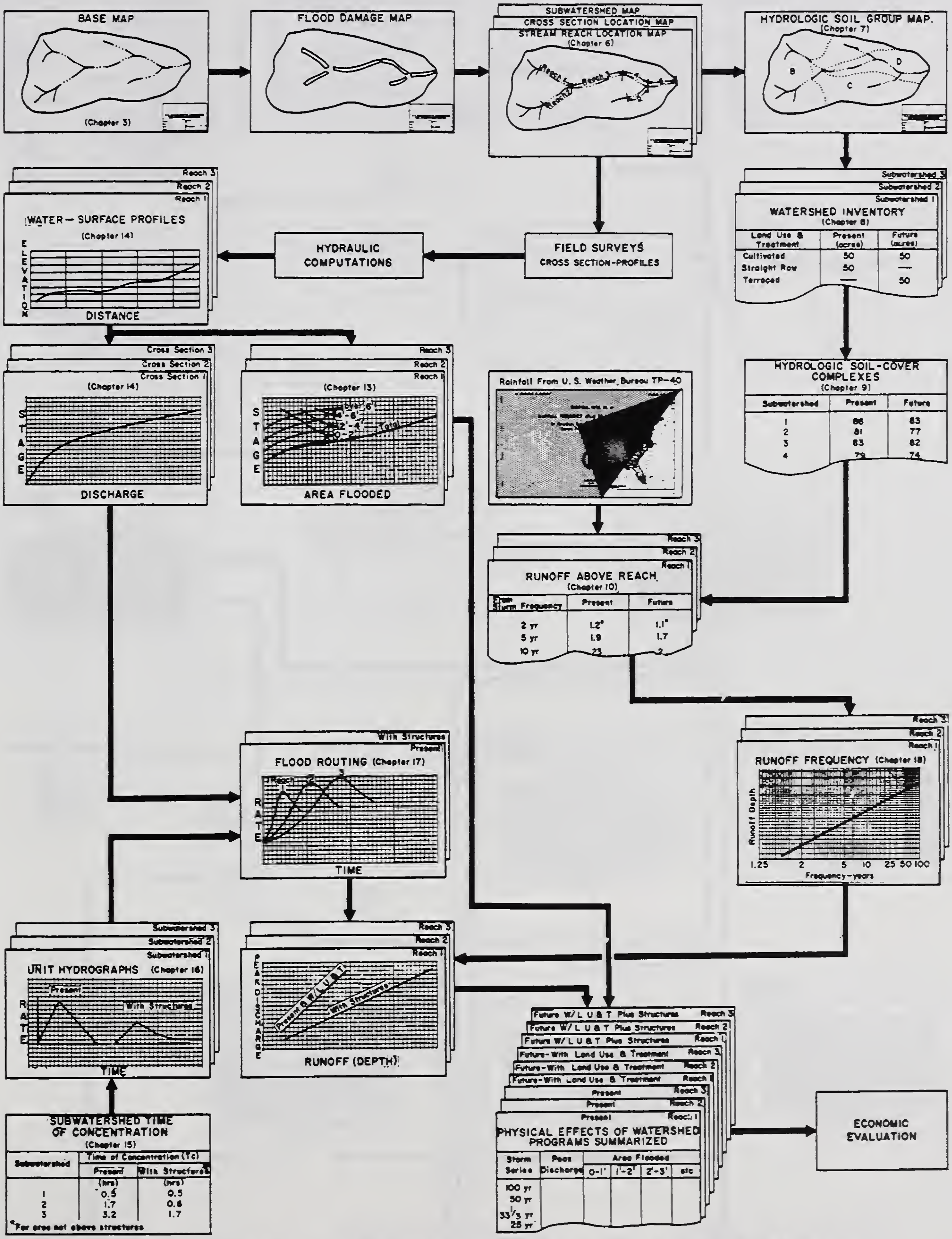

FIGURE 2.2-Hydrology of watershed evaluation with no stream flow or rainfall data available.

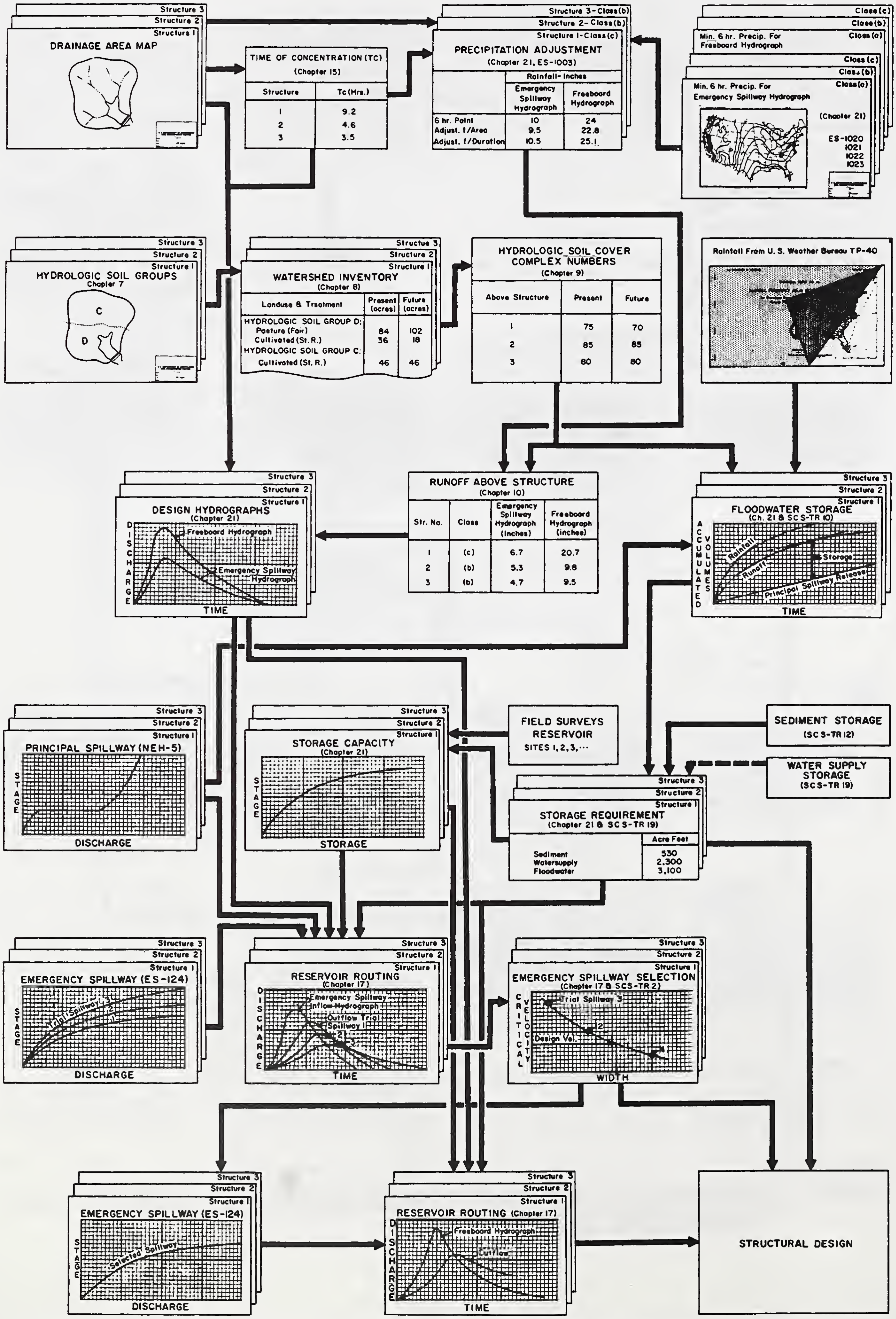

FIGURE 2.3-Design hydrology for storage and spillways in floodwater retarding structures.

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SECTION 4

HYDROLOGY

CHAPTER 3. PRELIMINARY INVESTIGATIONS

by

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SCS NATIONAL ENGINEERING HANDBOOK

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CHAPTER 3--PRELIMINARY INVESTIGATIONS

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CHAPTER 3. PRELIMINARY INVESTIGATIONS

A preliminary investigation (PI) is a brief study of a potential project in order to estimate whether a detailed investigation is justified. For a watershed-protection and flood-prevention project, the PI is mainly concerned with flood problems and their solutions. A work plan party makes a PI by examining available reports and data for a watershed, making a field reconnaissance, briefly evaluating their findings, and writing a concise report. SCS policy assigns the responsibility for selecting the degree of intensity of a PI to the State Conservationist. Once this degree is selected, the party modifies its procedures accordingly and makes the study. The hydrologist can make a valuable contribution to the study by supplying appropriate reports and data, by using suitable techniques on the problems, and by developing new techniques as the need arises.

Making the Preliminary Investigation

During a PI the hydrologist may be required to work in fields other than hydrology, therefore the following discussion covers the general conduct of a PI without undue emphasis on the hydrologic duties.

EXAMINATION OF AVAILABLE REPORTS AND DATA

The work plan party examines earlier reports made for the area in which the watershed is located. Such reports may contain material that will be useful in evaluating a potential project or in preparing the PI report. Bureau of Reclamation, Corps of Engineers, and State engineer reports may give applicable information or data. U.S. Weather Bureau, U.S. Geological Survey, and State university publications may provide appropriate data on rainfall and runoff. SCS soil-survey reports will provide soils and generalized cover information; the local SCS work unit conservationist should be consulted on the use of these reports because he can readily evaluate a wide range of information regarding a specific watershed in his area.

RECONNAISSANCE

A field reconnaissance gives the work plan party an opportunity to become familiar with the physical characteristics of the watershed, this familiarity being necessary to avoid making gross mistakes in evaluating the available information or in writing the report. Before making the reconnaissance the party obtains aerial photographs or available maps of the watershed. Suitable maps are detailed maps prepared by the SCS Cartographic Unit, SCS soil-survey maps, U.S. Geological Survey topographic sheets, or other similar maps. In addition to their use as direction-finders, the photographs or maps are used in the field for recording possible sites of floodwater-retarding structures or other measures, for designating areas of major flood-water or sediment damages, and for indicating areas requiring intensive study in a detailed investigation.

During the reconnaissance the hydrologist obtains estimates of Manning's n (chap. 14) and hydrologic soil-cover complexes (chaps. 7, 8, and 9) if such estimates are needed in the evaluation or report.

EVALUATION

The PI report is concerned with a potential project and its economic justification, so that magnitudes of rains or floods and similar data are introductory material of minor interest and the quantities of measures, damages, benefits, and costs are of major interest. The required quantities can generally be estimated by use of relationships developed from work plans or other studies already completed for the physiographic region in which the watershed lies. Some typical relationships are shown in figures 3.1 through 3.7. Relationships of this kind are used because the PI evaluation must be made in a relatively short time.

Figures 3.1 through 3.7 are not for general application to all watersheds because they were developed for particular areas and are valid only for these areas. But they illustrate principles that can be applied in developing relationships for other areas. All such relationships are empirical; this means that the lines of relation should not be extended very far beyond the range of data used in their construction. An example of the use of some of the relationships is given later in this chapter.

Figure 3.1 shows a relationship developed from data in work plans for projects containing floodwater-retarding structures but few channel improvements. The line of relation shows the minimum amount of watershed area that must be controlled by the structures in order for a project to be economically justified. For other areas the line of relation may be curved or have a different slope.

Figure 3.2 shows the average annual cost of a system of floodwater-retarding structures in relation to watershed area and percent of control for projects having few channel improvements. In this and other figures that show costs, the costs are valid only for the economic period for which they were originally applicable. An adjustment must be made for later periods.

Figure 3.3 shows another cost relationship, this one being for total cost of individual structures. The cost is related to the drainage area above a structure and to the land-resource area in which it lies.

Figure 3.4 shows the amount of flood-plain area in a watershed in relation to the product of total watershed area and average annual rainfall. Such a relationship is most effective for regions where the annual rainfall does not vary abruptly over the region.

Figure 3.5 shows the average annual direct damage for "present" conditions in relation to flood-plain area size and percent of cultivation. This figure was developed by means of a multiple-regression analysis (chap. 18). Similar relationships for other areas may be developed either by such an analysis or by a graphical method in which the data are plotted on log paper and a family of curves or straight lines is fitted by eye. Parameters other than "percent cultivated" may also be suitable. In relationships using damages in dollars, the damage estimates are valid only for the economic period in which they were originally applicable. An adjustment must be made for later periods.

Figure 3.6 shows another damage relationship for "present" conditions. This relationship applies within a region for which flood-frequency lines of the watersheds will have about the same slope when plotted on lognormal probability paper. For other regions the line of relation may have a different curvature. Figure 3.6 is used with a historical flood for which the frequency and total damage are known. For example, if a watershed in this region has had a flood with a 10-year frequency, then the curve gives a multiplier of 0.41; and if the total damage for that flood was $80,000, then the estimated average annual damage for the watershed is 0.41($80,000) = $32,800.

Figure 3.7 shows the average-annual-damage reduction due to use of a system of floodwater-retarding structures, in relation to the percent of the watershed controlled by the system. Lines of relation for different land-resource areas in a particular region are given. The reason for the variations by area is not specified in the original source of the figure but they may be due to one or more influences such as topography, soils, rainfall, or type of economy.

DISCUSSION. The chief requirement for such relationships is that they be conservatively developed. The lines of relation should be drawn in such a way that the estimates will be conservative; that is, the lines should tend to over-estimate costs and under-estimate benefits. If this is done, these relationships and other of a similar nature will be valuable working tools not only for PI's but also for river basin studies.

EXAMPLE. In this example it is assumed that figures 3.1, 3.2, 3.4, 3.5, and 3.7 apply to the land-resource area in which the problem-watershed lies. For this watershed it is necessary to estimate the benefit-cost ratio of a potential system of floodwater-retarding structures in order to state in the PI report whether further investigation of the project is worthwhile. The required data are as follows: the watershed is in land-resource area 4; the drainage area is 150 square miles, the average annual rainfall 24 inches, and the flood plain 60 percent cultivated. The following steps are taken to use the figures in estimating the ratio (all numerical estimates will be carried with as many digits as can be read from the figures and the rounding will be in the last step):

1. Estimate the minimum area that must be controlled to have an economically justified project. Enter figure 3.1 with the drainage area of 150 square miles and read an "area-controlled" of 80 square miles. In practice, the reconnaissance may show that more control can be obtained; if so, use the higher degree of control in the remaining steps.

2. Compute the percent controlled: 100(80/150) = 53 percent.

3. Estimate the average annual cost of the system. Enter figure 3.2 with the drainage area of 150 square miles and for 53-percent control read by interpolation an average annual cost of $36,000.

4. Estimate the amount of flood-plain area. First compute the product of drainage area and average annual rainfall: 150(24) = 3,600. Next enter figure 3.4 with this product and read a flood-plain area of 5,200 acres.

5. Estimate the average annual direct damages. Enter figure 3.5 with the flood-plain area of 5,200 acres and at the line for 60-percent cultivated read damages of $75,000.

6. Estimate the reduction in average annual direct damages. Enter figure 3.7 with the percent controlled from step 2 and at the line for land-resource area 4 read a reduction of 73 percent.

7. Compute the estimated benefits. Use the average annual direct damages in step 5 and the percent reduction in step 6: (73/100)($75,000) = $54,750.

8. Compute the estimated benefit-cost ratio. Use the benefit in step 7 and the cost in step 3. The ratio is $54,750/$36,000 = 1.52. Round to 1.5, which is the required estimate for this example.

In this example the benefit-cost ratio is favorable and a recommendation can be made in the PI report that further investigation is justified. If the ratio happens to turn out slightly unfavorable, it may still be desirable to recommend further investigation because the short-cut procedure is conservative and a detailed investigation may show that the project is economically feasible. But if the ratio is very unfavorable it is not likely that a detailed investigation can improve it, and alternative project measures need to be considered instead.

Report

The general format of a PI report will not be discussed here because each State establishes its own pattern. Usually the hydrology in the report is merely descriptive but, if it is necessary to show hydrographs of present and future (with project) flows in the report, the hydrologist can find short-cut methods of estimating runoff amounts in chapter 10 and of constructing hydrographs in chapters 16 and 17.

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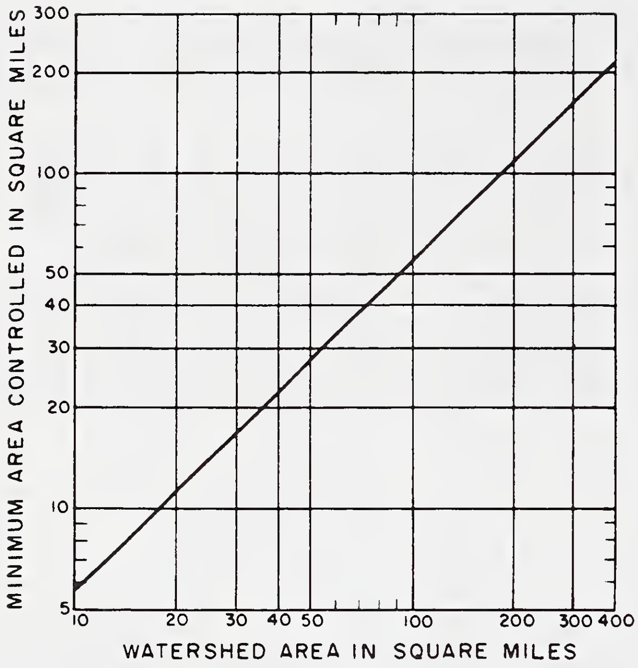

Figure 3.1.--Typical relationship for estimating the minimum amount of area it is necessary to control by floodwater-retarding structures.

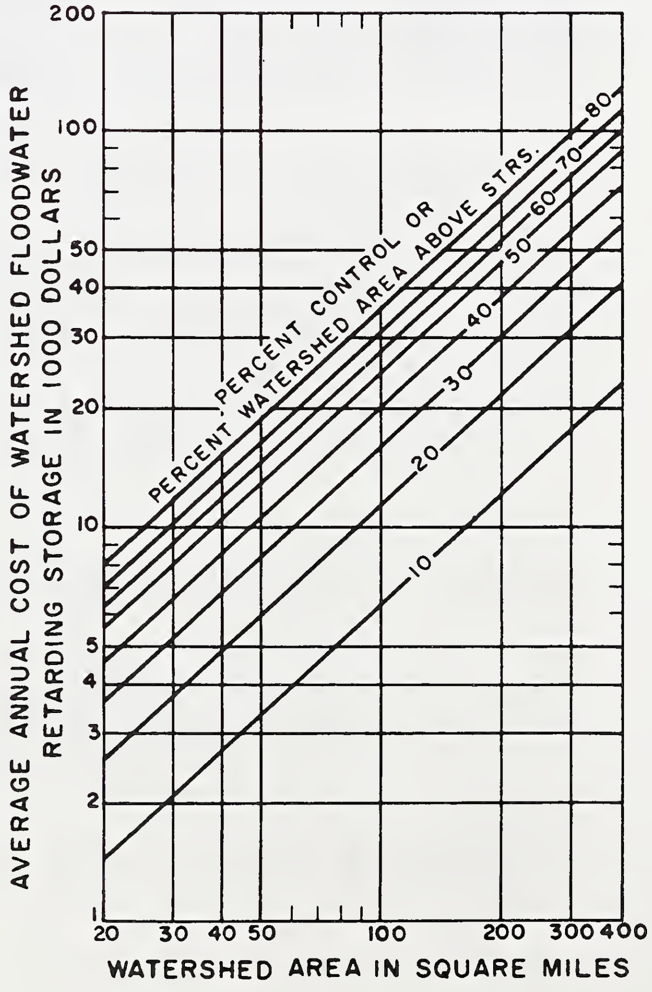

Figure 3.2.--Typical relationship for estimating the average annual cost of a system of floodwater-retarding structures.

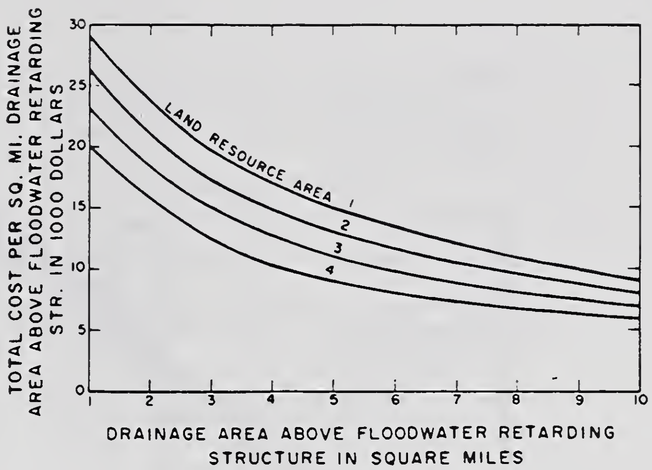

Figure 3.3.--Typical relationship for estimating the total cost of a system of floodwater retarding structures.

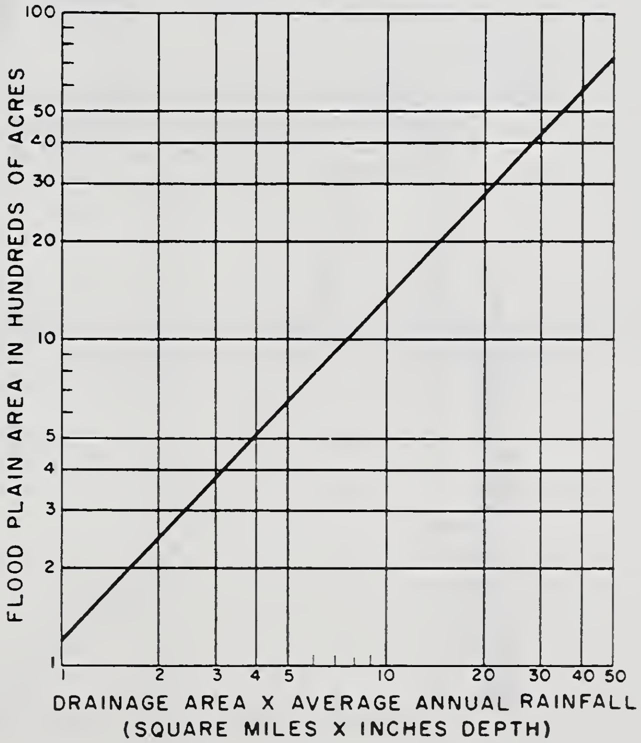

Figure 3.4.--Typical relationship for estimating the amount of flood-plain area in a watershed.

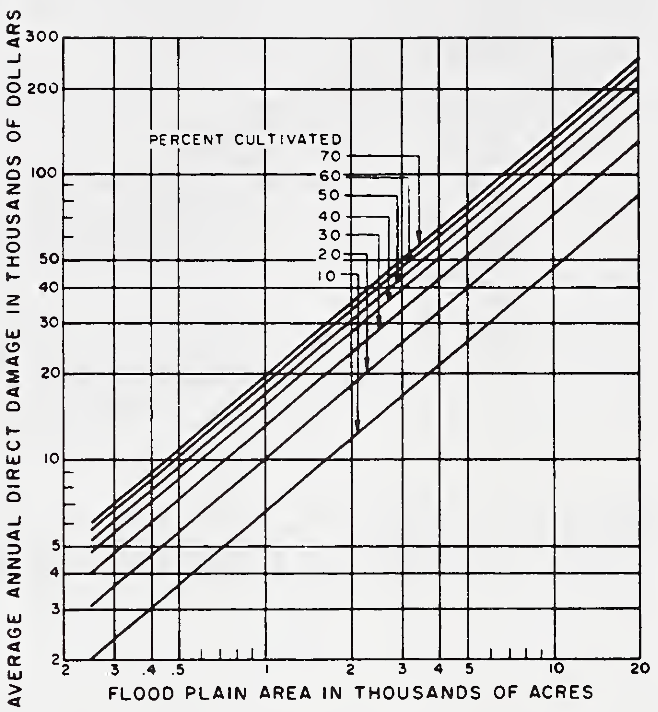

Figure 3.5.--Typical relationship for estimating the average annual direct damage.

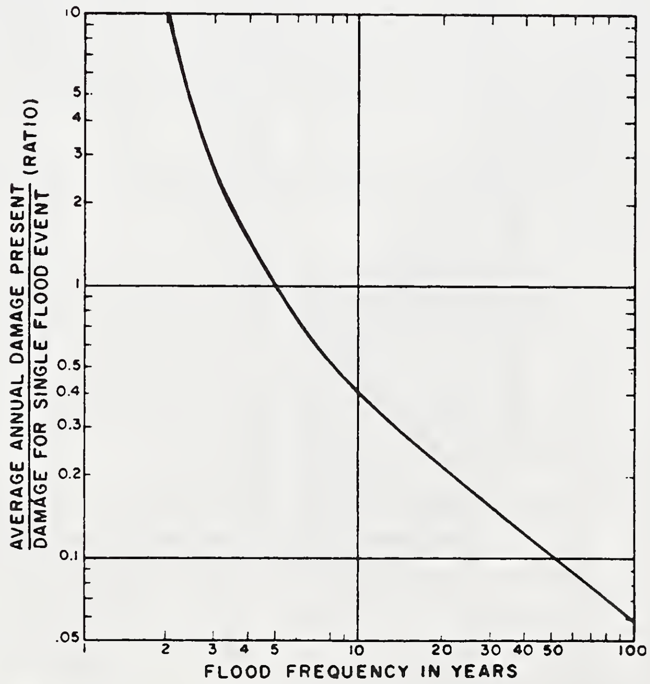

Figure 3.6.--Typical relationship for estimating present average annual flood damages.

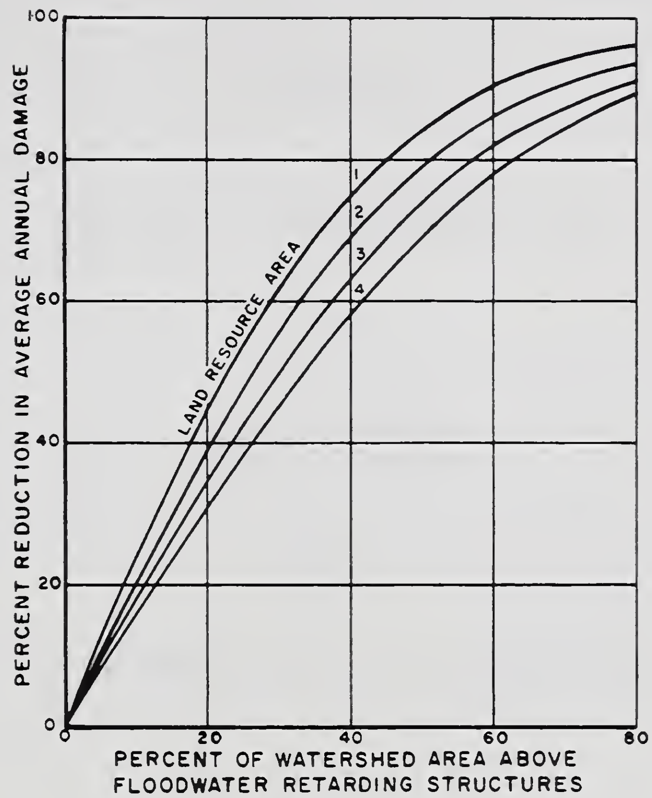

Figure 3.7.--Typical relationship for estimating the reduction in average annual flood damages.

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SECTION 4

HYDROLOGY

CHAPTER 4. STORM RAINFALL DATA

by

Victor Mockus
Hydraulic Engineer

1964

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SECTION 4

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CHAPTER 4--STORM RAINFALL DATA

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CHAPTER 4. STORM RAINFALL DATA

This chapter gives a brief account of the sources, variability, and preparation of rainfall data used for estimating storm runoff (chapter 10) and for designing floodwater-retarding structures (chapter 21). The account also applies to monthly and annual rainfall. Probable maximum precipitation is discussed in chapter 21.

Sources of Data

The storm rainfall data used in this handbook are daily total amounts or storm totals as measured at rain gages, or total amounts for specified durations as found in statistical studies made by the U. S. Weather Bureau. The choice of data is due to their availability on a national basis, and it was for use of such data that the runoff estimation method of chapter 10 was developed.

A comprehensive account and bibliography of rain gage designs, installations, and measurement research is given in "Precipitation Measurements Study" by John C. Kurtyka, Report of Investigation No. 20, 178 pp, Illinois State Water Survey Division, Urbana, Ill., 1953. Gages used in the U. S. Weather Bureau network are described in "Instructions for Climatological Observers," U. S. Weather Bureau Circular B, pp 76, 11th edition, 1962; U. S. Government Printing Office, Washington, D. C. 20042, price $0.50.

PUBLISHED DATA

Daily amounts of rainfall measured at gages in their official network are published by the U. S. Weather Bureau in monthly issues of "Climatological Data" for each State. The times of daily measurement vary,

as indicated by footnotes in the publications. Storm totals and durations can be obtained from the Weather Bureau's "Hourly Precipitation Data" for each State. Other Federal and State agencies and universities publish rainfall data at irregular intervals, often in a special storm report or a research paper.

UNPUBLISHED DATA

Various Federal and State agencies will sometimes make field surveys after an unusually large storm to collect "bucket-survey" data, which are measurements of rainfall caught in buckets, watering troughs, bottles, and similar containers. Ordinarily these data are used to give more detail to rainfall maps based on standard-gage data. Generally when the catch of a "bucket gage" exceeds the catch of the nearest standard gage by more than about 25 percent, the bucket gage catch should be carefully evaluated. Data from bucket surveys are generally not published but are available in the offices of the gathering agency.

Narrow-bore tubes of the kind used by many farmers and ranchers have been shown to give results almost equal to those from standard gages. Tube gages must be properly exposed and serviced to obtain such results. Most farmers and ranchers keep a daily or storm record of catches.

Many newspaper offices, banks, water-treatment plants, and other municipal offices collect measurements at their own gages and keep daily records.

PUBLISHED RAINFALL-DATA ANALYSES

In many kinds of hydrologic work it is unnecessary to use actual rainfall data since published analyses of data provide the required information in more usable form. The following published rainfall-data analyses were made by the U. S. Weather Bureau in cooperation with SCS:

1. "Rainfall Frequency Atlas of the United States", U.S. Weather Bureau Technical Paper No. 40; 115 pages; price $1.75. Includes all States except Alaska and Hawaii.

2. "Generalized Estimates of Probable Maximum Precipitation and Rainfall-Frequency Data for Puerto Rico and Virgin Islands," U. S. Weather Bureau Technical Paper No. 42, 94 pages, price $0.50.

3. "Rainfall-Frequency Atlas of the Hawaiian Islands," U. S. Weather Bureau Technical Paper No. 43, 60 pages, price $0.40.

4. "Probable Maximum Precipitation and Rainfall-Frequency Data for Alaska", U. S. Weather Bureau Technical Paper No. 47, 69 pages, price $1.00.

These publications are available from the U. S. Government Printing Office, Washington, D. C. 20042, at the prices shown.

Watershed Rainfall

In watershed work it is often necessary to know the average depth of storm rainfall over an area. The average depth may be found in various ways, depending on the kind of data being used. If the rainfall amount is taken from one of the USWB Technical Papers it is for a point at a specific locality, and the point-area relationship given in the Paper is used to estimate the average depth over an area in that locality. Examples in the Papers illustrate the procedure. It is more difficult to obtain an average depth from data of one or more rain gages because the results are influenced by the number and locations of gages and the storm size. Methods of using such data are given in this section.

METHODS OF ESTIMATING AVERAGE DEPTHS

Use of One Gage

How well the rainfall measured at one gage will represent the average depth over an area depends on (i) Distance from the gage to the center of the area, (ii) Size of the area, (iii) Kind of rainfall amounts being used, (iv) Topography of the locality, and (v) Characteristic storm pattern of the locality. The effects of the first three influences will be illustrated here by means of figure 4.1, the fourth will be discussed in Orographic Influences, and the fifth in connection with figure 18.--, though it is implied in the relationships in figure 4.11 and whenever a comparison is made between storms in different localities.

The effect of distance is seen in (a) and (b) of figure 4.1. In (a) a single gage is located near the center of a 0.75-square-mile watershed and the storm rainfall catches at the gage are seen to be quite

close to those of the watershed averages, which were determined using a dense network of gages. But in (b), where the gage is located 4 miles from the watershed boundary, the storm rainfall catches at the gage often differ significantly (in the statistical sense) from the watershed averages. A similar effect is found when the area of application is increased as in (c), where the gage is near the outlet of a 5.4-square-mile watershed.

There is a close correspondence of gage catches and area averages when the rainfall amounts being used are sums of catches, such as monthly or annual rainfalls, because the errors for single storms tend to offset each other. The gage and watershed of (c) are used in (d) where annual rainfalls are plotted. The differences between gage and watershed amounts are seen to be relatively smaller than in (c). There will also be a close correspondence of gage and area amounts if the storm rainfalls are used with the methods of chapter 18 to construct frequency lines for gage and area. The correspondence occurring then is for amounts having the same frequency.

These examples show that the use of one gage brings up the question of how much error is permissible in the area estimate. This subject is discussed further under Accuracy.

Isohyetal Method

The spacing of gages in a network over an area is seldom uniform enough for taking an average of the gage catches as the area average. Isohyetal maps are used, with networks of any configuration, to get area averages or for studies of rainfall distributions. An isohyet is a line connecting points of equal rainfall depth and the map is made by drawing the lines in the same manner that contour lines are drawn on topographic maps, using the gage locations as data points. The isohyetal method can be used for hilly or mountainous areas when supplementary graphs, like that of figure 4.8, are available for the locality.

Figure 4.2 shows a simple application of the isohyetal method to a research watershed in Nebraska. The watershed average depth can be obtained as follows: If the isohyetal pattern is fairly even across the watershed as in (c), a point at the center of the area gives the average depth. The estimate made using point A in (c) is 1.59 inches. If the isohyetal pattern is not even, divide the watershed into parts for which the pattern is sufficiently uniform, make an estimate for each part, and get the watershed average by weighting or averaging the amounts for the parts.

A denser network gives the more complicated isohyetal map shown in (d), where the regular network on this research watershed is used for the storm also shown in (c). There is an important change in depth on parts of the watershed but the watershed average is 1.61 inches, not a significant improvement in accuracy over the estimate in (c). A particular network may therefore be excessively close for one kind of estimate at the same time it is too open for another kind. The relative error of an area average obtained through use of a network can be estimated as shown under Accuracy.

Thiessen Method

Another method of using a rain gage network for estimating watershed average depths is the Thiessen method, especially suitable for electronic computer routines. In the Thiessen method the watershed area is divided in subareas, using rain gages as hubs of polygons. The subareas are used to determine ratios which are multiplied by the subarea rainfall and summed to get the watershed average depth. The polygonic diagram is constructed as shown in figure 4.3 (a) and (b), and the Thiessen weights are computed. These weights are the ratio of the gage's polygon area divided by the area of the entire watershed as in (c). Watershed average depths are computed as shown in table 4.1 in which the storm of figure 4.2 is used. If a gage is added or removed from the network, a new diagram is drawn and new weights are computed.

The Thiessen method is not used to estimate rainfall depths of mountainous watersheds since elevation is also a strong factor influencing the areal distribution (see Orographic Influences).

Accuracy

Regardless of the method used the accuracy of the resulting rainfall estimate depends mainly on the distance between a gage and the point of application of the estimate. In mountainous areas the vertical distance may be more important than the horizontal, but for flat or rolling country only the horizontal distance matters. For a network both distance and arrangement of gages affect the accuracy. It is generally assumed that the catches at gages are exact measurements. This is seldom true because wind or splash effects can occur even when the gage is properly located, and there is always the possibility of error in reading the catch. But unless special studies at a gage site have been made the measurement errors are generally ignored.

Figure 4.4 is a diagram used for estimating the range of error likely

Table 4.1.--Watershed rainfall depth by the Thiessen method.

Rain gage	Measured rainfall	Thiessen weight		Weighted rainfall
	Inches			Inches
A	1.40	0.407		0.570
B	1.54	.156		.240
C	1.94	.437		.848
			Sum:	1.658

Watershed weighted rainfall depth is 1.658 inches, which is rounded off to 1.66 inches.

to occur nine times out of ten when the catch at a single gage is used as a depth for a location some distance away. It is modified from information in "Rainfall Relations on Small Areas in Illinois", by F. A. Huff and J. C. Neill, Bulletin 44, Illinois State Water Survey, Urbana, Illinois, 1957. Equation 5 on page 31 of this reference was modified to give results on a 10-percent level of significance. Horizontal distance is used, so that the diagram does not apply in mountainous areas. The following examples show how the diagram can be used.

Example 4.1.--The storm rainfall depth at a gage is 3.5 inches. What rainfall depth is likely to have occurred, with a probability of 0.9 (nine chances out of ten), at a point 5 miles away from the gage?

1. Enter figure 4.4 with the distance of 5 miles and at the intersection of the 3.5-inch line (by interpolation), read a "plus error" of 2.1 inches.

2. Compute a minus error as one-half of the plus error. This gives 2.1/2 = 1.05 inches. Round off to 1.1 inches.

3. Compute the range of rainfall likely to have occurred, nine chances out of ten. The limits are: 3.5 + 2.1 = 5.6 inches, and 3.5 - 1.1 = 2.4 inches. Therefore, when the gage has a catch of 3.5 inches there is a probability of 0.9 (nine chances out of ten) that the rainfall depth at a point 5 miles away from the gage is between 5.6 and 2.4 inches.

In step 2 of Example 4.1 the minus error is taken as one-half the plus error. This is an approximation, but it will be seen in the next example,

and the discussion following, that the approximation generally applies. The graphs of figure 4.5 will be used. The plottings on this figure show the variation to be expected when data at one gage are used to estimate the rainfall depth at a distant point.

Example 4.2.--Rain gages B28R and G42R, on the Agricultural Research Service watershed in Webster County, Nebr., are 4.3 miles apart. Given any storm rainfall of 0 to 4 inches depth at G42R, compute the range of error to be expected if the rainfall at B28R is to be estimated from that at G42R. Use figure 4.4. Compare the computed range with the plotting of actual data for the two gages.

1. Plot a line of equal values, which is the middle line on figure 4.5 (a).

2. Select three magnitudes on the G42R depth scale, these magnitudes to be used with figure 4.4. For this example the selected magnitudes are 1, 2, and 4 inches.

3. Enter figure 4.4 with the distance of 4.3 miles and at the intersections of the 1-, 2-, and 4-inch rainfall lines read plus errors of 1.15, 1.50, and 2.15 inches respectively.

4. Compute the minus errors. These are 0.58, 0.75, and 1.08 inches.

5. Plot the plus-error and minus-error lines as shown on figure 4.5 (a). The plotted points that are shown are for actual measurements at the gages. Three points of the gaged data fall outside the error range, so that the expected error for this pair of gages is somewhat less than predicted by figure 4.4.

One advantage in using figure 4.4 is that when a rainfall estimate is to be made for some distant point, the error lines can be drawn in advance to give an idea of the value of the estimate. Note that the percent error decreases as the rainfall amount increases. Error lines have also been drawn for (b), (c), and (d) of figure 4.5, using the method of Example 4.2, as a further check on figure 4.4. In each of the plottings a different number of points falls outside the error lines but on the average only 10 percent should be outside. This is confirmed by the following computation:

Figure 4.5:	(a)	(b)	(c)	(d)	Total
Number of Points:	91	35	7	20	153
Number outside lines:	3	10	0	3	16
Percent outside lines:	3.3	28.6	0	15.0	10.46

Figure 4.6 serves the same purpose for an area that figure 4.4 serves for a point. In using figure 4.6 it will be necessary to determine the number of gages on the watershed. The number is seldom clearly evident, as the typical examples of figure 4.7 show. In (a) of this figure the gage network ABC would be used for an isohyetal map or in computing Thiessen weights. The watershed average rainfall depth estimated from an isohyetal map based on the use of ABC would be more accurate than if based on BC. Therefore it would not be correct to say there are only two gages "on" the watershed when figure 4.6 is used. In (b), however, although all six gages of the network DEFGHI are physically within the watershed, gages DEFG are much too close together (by comparison with the remaining gages) to be considered as individual gages. In (c) where gages JKLMNP have varying distances between adjacent gages, it is still more difficult to say how many gages are "in" the watershed. With the case shown in (d), where the network QRST is completely outside the watershed (but still usable for construction of an isohyetal map) any decision on number of gages "in" the watershed would be arbitrary.

Therefore, figure 4.6 should be used without spending much time on deciding how many gages are applicable. The examples that follow will illustrate what can be done even with the extreme cases of figure 4.7. Note that figure 4.6 gives an average error that is of the same magnitude plus and minus, in this respect differing from figure 4.4.

Example 4.3.--Assuming that the watershed of figure 4.7(a) has a drainage area of 200 square miles and an average annual rainfall of 35 inches, find the average error of estimate when the watershed average depth is 4.5 inches.

Figure 4.6 is used first with a network of two, then of three gages, and the results are compared. The figure shows that a 2-gage network gives an error of about 13 percent, and a 3-gage network an error of about 8 percent. In either case the error is relatively small.

Example 4.4.--The standard error in percent (see chapter 18) can be estimated, if it is needed, by taking 1.5 times the average error. For example 4.3 the computations are:

2-gage network, standard error = 1.5(13) = 19.5 %
3-gage network, standard error = 1.5(8) = 12.0 %

Example 4.5.--The size of the watershed itself can have no bearing on the watershed average rainfall depth when the network is that of figure 4.7(d). In such cases the area of the polygon formed by the network QRST is used in figure 4.6. If the watershed average annual rainfall is 35 inches and the network polygon area is 375 square miles then, for a 5-inch rain, figure 4.6 gives an estimate of about 8 percent error. This is for the area of the polygon and,

presumably, for any watershed within it. It is reasonable to expect that the smaller the watershed the larger the error will be, but there is no way to determine this on the basis of present information.

It should be evident that figure 4.6 must be used with some imagination and that it gives only rough approximations. And for cases like the networks in (b) and (c) of figure 4.7 neither the number of gages to be used nor the area of applicability is easy to define. Despite these limitations, figure 4.6 has the worthwhile function of keeping the hydrologist aware of the range of error possible in his calculations.

Use of Published Analyses

Methods of using the rainfall information in the USWB Technical Papers are given in the papers themselves and additional examples will be found in chapter 21. Figures 4.4 and 4.6 do not apply to rainfall information from these papers. A discussion of the errors involved in use of the depth-duration-frequency maps of those papers will be found on Pages 4 and 5 of USWB Technical Paper 40, where the following statement is made:

> "Evaluation.--In general, the standard error of estimate ranges from a minimum of about 10%, where a point value can be used directly as taken from a flat region of one of the 2-year maps, to 50% where a 100-year value of short-duration rainfall must be estimated for an appreciable area in a more rugged region."

OROGRAPHIC INFLUENCES

In hilly or mountainous country, rainfall catches are influenced by physiographic variables, both local and distant. Some of these are (1) elevation or altitude, (2) local slope, (3) orientation of the slope, (4) distance from the moisture source, (5) topographic barriers to incoming moisture, and (6) degree of exposure, which is defined as "The sum of those sectors of a circle of 20-mile radius centered at the station, containing no barrier 1,000 feet or more above station elevation, expressed in degrees of arc of circle (azimuth)," (from "The Analysis of Precipitation Data", by W. E. Hiatt; Vol. IV, The Physical and Economic Foundation of Natural Resources Series.).

In the ordinary watershed study it is seldom possible to determine the influences of all these variables. When a special study is needed for a project, the SCS hydrologist can apply to the Chief,

Hydrology Branch, who can make arrangements for a cooperative study by the Weather Bureau.

When extreme accuracy is not required, the effects of elevation can be estimated from elevation and rainfall data alone, if other influences can be held constant within zones. The area or watershed is divided into zones for which influences other than elevation are believed to be fairly constant, and a graphical relation of elevation versus rainfall is developed for each zone. The relation for a zone is used with elevations in that zone to locate isohyetals on an isohyetal map, with measured catches being firm data. Figure 4.8 is an example where orientation (coast side, desert side) is used to define zones. The relation for "coast side" is seen to be satisfactory, but the one for "desert side" appears to be affected by the influences that were ignored. A somewhat different example using orientation is given in Weather Bureau Technical Paper 47.

Antecedent Rainfall

Rainfalls in antecedent periods of 5 to 30 or more days prior to a storm are commonly used as indexes of watershed wetness. An increase in an index means an increase in the runoff potential. Such indexes are only rough approximations because they do not include the effects of evapotranspiration and infiltration on watershed wetness. Therefore, it is not worthwhile to try for great accuracy in computing the index described below.

ANTECEDENT MOISTURE CONDITION

The index of watershed wetness used with the runoff estimation method of chapter 10 is Antecedent Moisture Condition (AMC). Three levels of AMC are used:

AMC-I. Lowest runoff potential. The watershed soils are dry enough for satisfactory plowing or cultivation to take place.

AMC-II. The average condition.

AMC-III. Highest runoff potential. The watershed is practically saturated from antecedent rains.

The AMC can be estimated from 5-day antecedent rainfall by the use of table 4.2, which gives the rainfall limits by season categories. The

table is adapted from material developed by the Fort Worth EWP Unit. The rainfall limits are plotted as boundary points for the AMC groups in figure 4.9, which illustrates the linear character of the index. No upper limit is intended for AMC-III, as table 4.2 shows. The limits for "dormant season" apply when the soils are not frozen and there is no snow on the ground.

The 5-day rainfall amount used with table 4.2 or figure 4.9 is a simple total. For example, if the AMC for a watershed is to be estimated for the date of June 8, which is in the growing season, and if the rain for the preceding five days is:

June 3	June 4	June 5	June 6	June 7
0.10	0	0.35	0.15	0.72

then the total rainfall of 1.32 inches, used with the "growing season" column of table 4.2, shows the appropriate moisture group to be AMC-I. Additional examples of the use of table 4.2 are given in chapter 10.

Storm Duration

The total duration of a storm is used in estimating a peak rate of runoff or in developing a hydrograph. The duration is always known for a design storm, but for natural storms, such as those used in some methods of watershed evaluation, the duration may be difficult to determine. Methods of estimating the duration of natural storms will be briefly discussed.

NATURAL STORMS

Durations of specific actual storms can generally be estimated to the nearest hour by use of Weather Bureau publications of hourly precipitation data. With these data, or even with instrument charts from a recording gage, it is often difficult to decide on the beginning or ending times of a storm. Furthermore, if there are periods of no rain within the storm, the duration may need to be arbitrarily defined. The problem of hydrograph construction is simplified by using storm increments and, in general, this is the best way of using natural storms (for hydrograph construction in this manner, see chapter 16).

Table 4.2.--Seasonal rainfall limits for AMC.

AMC group	Total 5-day antecedent rainfall	
	Dormant season	Growing season
	Inches	Inches
I	Less than 0.5	Less than 1.4
II	0.5 to 1.1	1.4 to 2.1
III	Over 1.1	Over 2.1

Figure 4.10 illustrates a typical natural storm for which the storm duration must be arbitrarily defined. The figure shows the accumulated runoff occurring when the runoff curve numbers of 100, 80, and 70 are applicable. Note that the duration of excessive rainfall, which is the rainfall producing the runoff, is always less than the storm duration except when runoff is 100 percent. Since the duration of excessive rainfall is the correct duration to use with peak rate equations, such equations will be more successful with natural storms that are brief and intense. The hydrologic design methods of chapter 21 have been developed to account for the initial abstraction,so that duration of excessive rainfall is used.

Effective Duration (D_e)

When standard gage data are used in a watershed project evaluation, the storm durations will usually be unknown. An approximate duration for use with all the storms can be estimated using figure 4.11, which shows the relation between average annual rainfall and an "effective duration". The gage rainfalls are used as if they had fallen in D_e hours. The plotted points on figure 4.11 were obtained by different methods. The method used in obtaining the St. Louis, Mo. point will be described, since it best illustrates the significance of D_e.

The hourly records of precipitation at St. Louis, Mo. were used to estimate, to the nearest hour, the duration of each rain in the period March 1920 through December 1929. The form shown in table 4.3 was used to tally the durations. If there was no rain for one or more hours, the duration was decreased by the same number of hours. The preponderant number of rains was continuous. After completing the tabulation, the number of tallies was accumulated as shown in column 4. The median number of items is 162.5 (the grand total is an even number, and the median is obtained

Table 4.3.--Duration of daily rainfalls at St. Louis, Mo. for the period March 1920 through December 1929.

Duration	Tallies	No. of tallies	Accumulated tallies
Hours			
1	𝍸 𝍸 𝍸	15	15
2	𝍸 𝍸 𝍸 𝍸 𝍸 |	26	41
3	𝍸 𝍸 𝍸 𝍸 𝍸 𝍸	30	71
4	𝍸 𝍸 𝍸 𝍸 𝍸 |||	28	99
5	𝍸 𝍸 𝍸 𝍸 ||	22	121
6	𝍸 𝍸 𝍸 𝍸 ||||	24	145
7	𝍸 𝍸 𝍸 𝍸 ||	22	167*
8	𝍸 𝍸 𝍸 𝍸 𝍸	25	192
9	𝍸 𝍸 𝍸 𝍸 𝍸	25	217
10	𝍸 𝍸 𝍸 |||	18	235
11	𝍸 𝍸 𝍸 |	16	251
12	𝍸 ||||	9	260
13	𝍸 ||	7	267
14	𝍸 ||	7	274
15	𝍸	5	279
16	𝍸 |	6	285
17	𝍸	5	290
18	𝍸	5	295
19	|||	3	298
20	||||	4	302
21	|||	3	305
22	𝍸	5	310
23	||||	4	314
24	𝍸 𝍸	10	324

* Median is within this group.

by averaging the two numbers adjacent to it). The duration D_e was found by interpolating as follows:

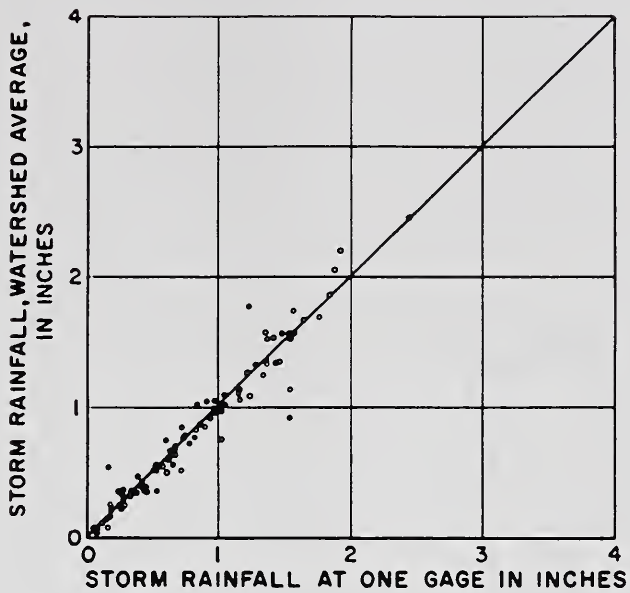
$$D_e = 6 + (7 - 6) \frac{162.5 - 145}{167 - 145} = 6.8 \text{ hours}$$

The average annual precipitation at St. Louis for this period was 38.45 inches. This amount is plotted versus the D_e as shown in figure 4.11. In using this D_e, the daily catches at St. Louis are assumed to have fallen in 6.8 hours. Ratios obtained from Weather Bureau Technical Paper 40 do not apply to a D_e because the tabulations used for TP-40 were of another kind.

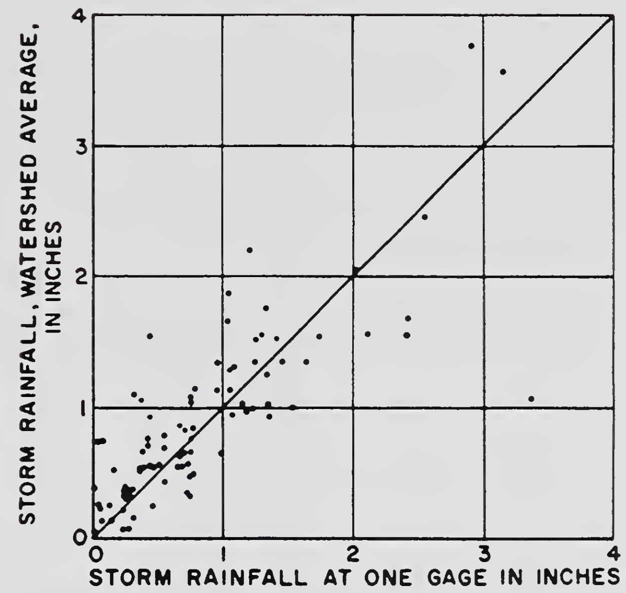
* * * *

Addendum Regarding Figures 4.4 and 4.6.--These charts can be applied to rainfall data in mountainous areas in this way: in those areas the error will always be <u>larger</u> than that shown by either chart.

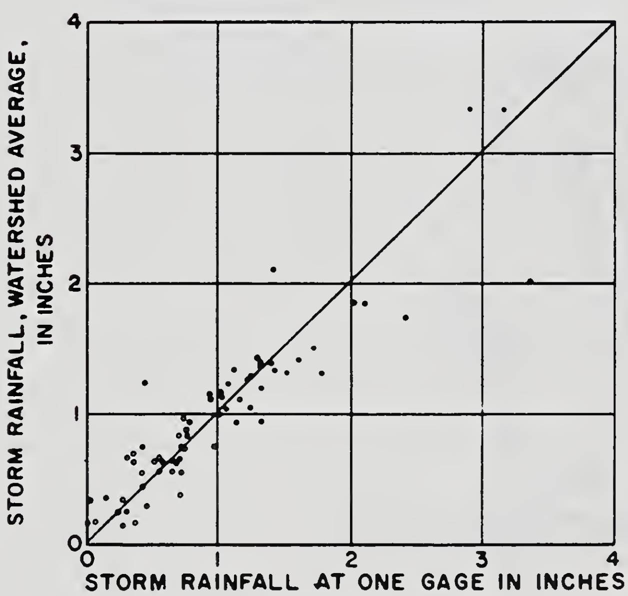
* * * *



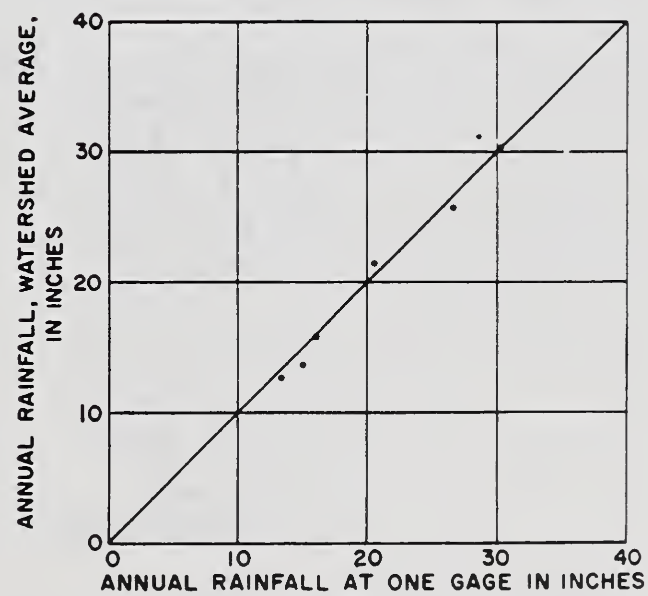
(a) WATERSHED AREA IS 0.75 SQUARE MILES AND GAGE IS NEAR THE CENTER.



(b) WATERSHED AREA IS 0.75 SQUARE MILES AND GAGE IS 4 MILES OUTSIDE THE WATERSHED BOUNDARY.



(c) WATERSHED AREA IS 5.45 SQUARE MILES AND THE GAGE IS ON THE BOUNDARY.



(d) WATERSHED AREA IS 5.45 SQUARE MILES AND THE GAGE IS ON THE BOUNDARY.

Figure 4.1.--Errors due to use of catches at one gage as estimates of watershed average rainfall.

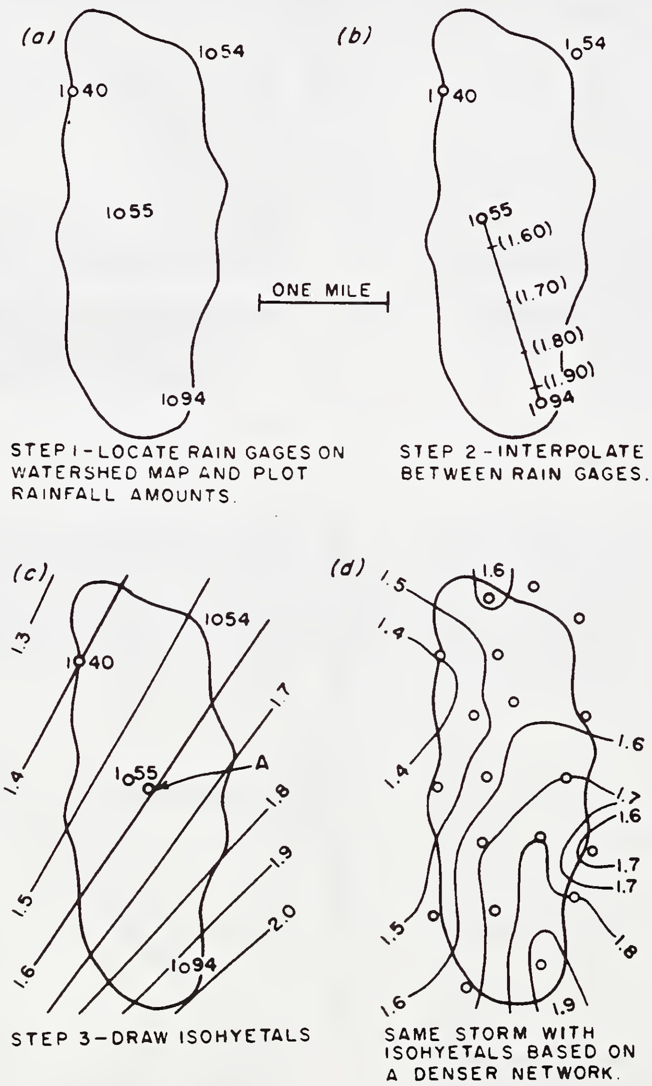


Figure 4.2.--Steps in construction of an isohyetal map. Circles used as decimal points also denote rain gages. The two lower maps illustrate the variations due to use of different networks of gages.

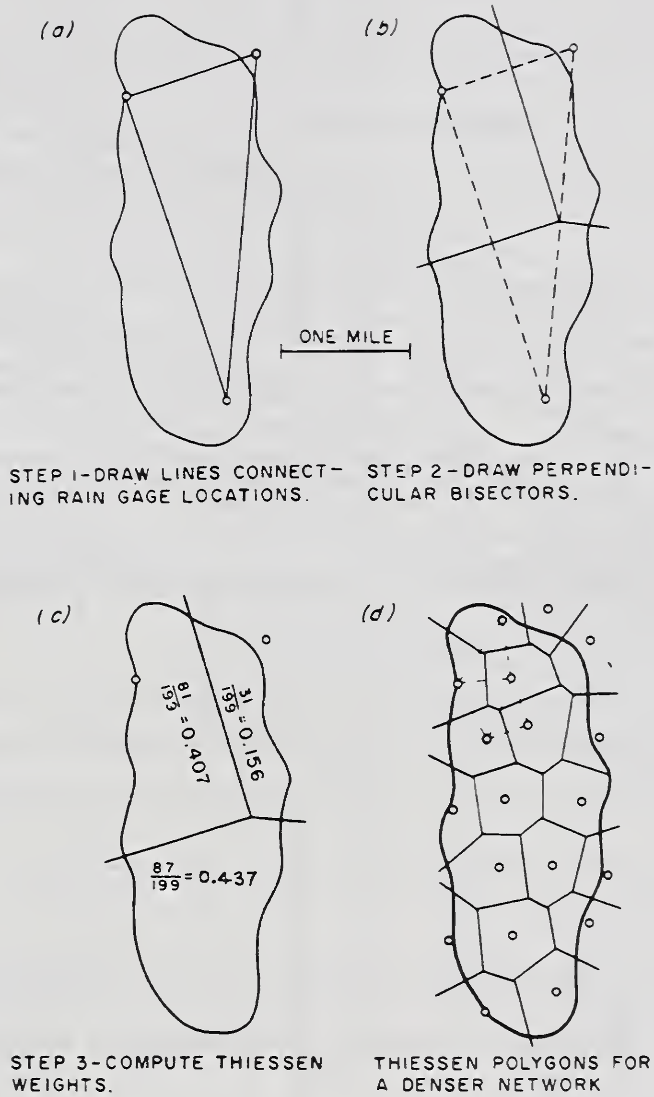


Figure 4.3.--Steps in the determination of Thiessen weights. The two lower maps illustrate the variations in polygons due to use of different networks of gages.

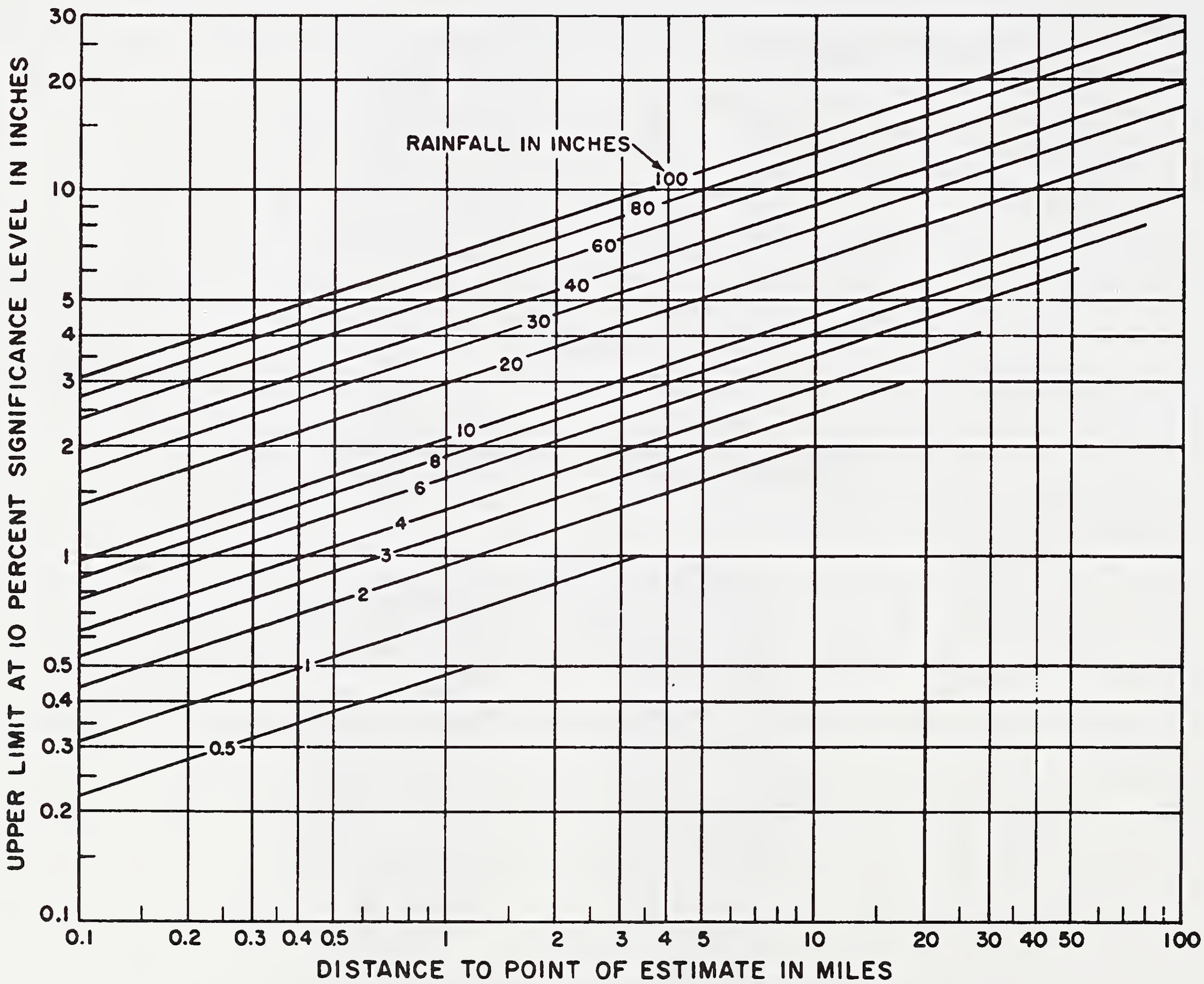


Figure 4.4.--Graph for estimating the upper (positive) increment of error in transposed rainfall amounts. The 10-percent level of significance applies to this increment. The lower (negative) increment is taken as 1/2 the upper. The graph does not apply to rainfalls in mountainous areas.

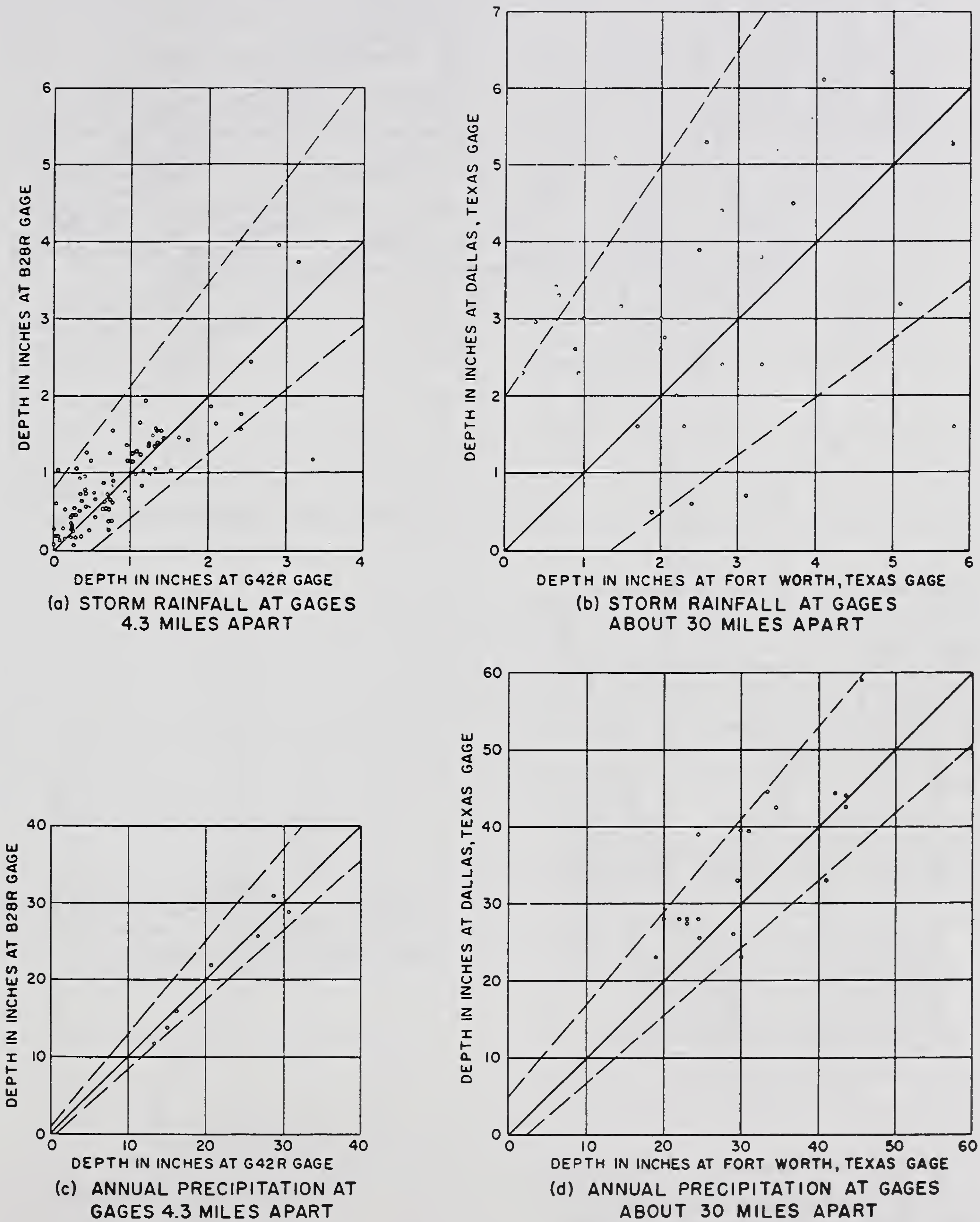


Figure 4.5.--Applications of figure 4.4. The dashed lines show the range in rainfall to be expected, 90 percent of the time, at a distant location (ordinate) when the rainfall amount at a gage (abscissa) is transposed. The plotted points are actual measurements at the distant and gage locations.

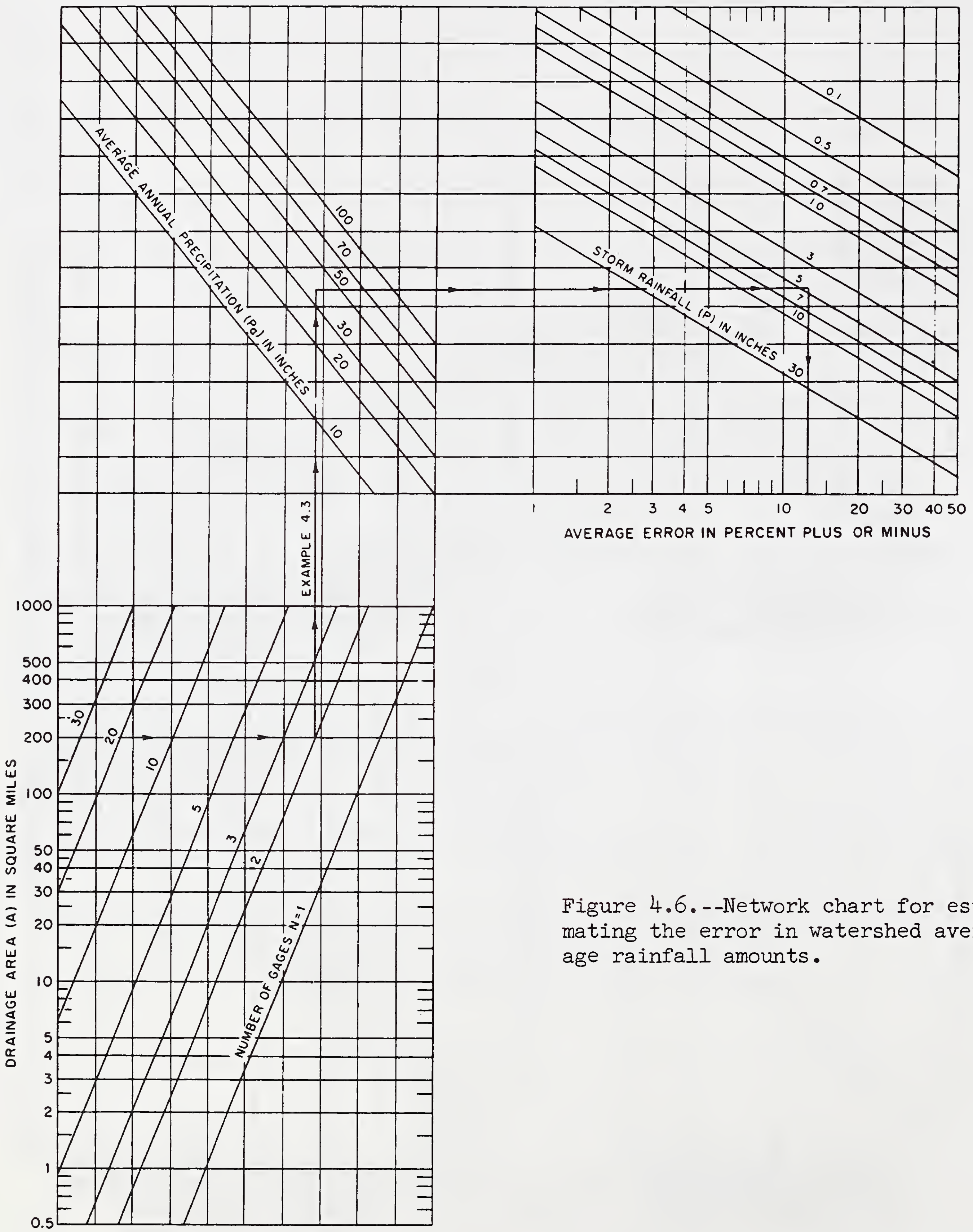


Figure 4.6.--Network chart for estimating the error in watershed average rainfall amounts.

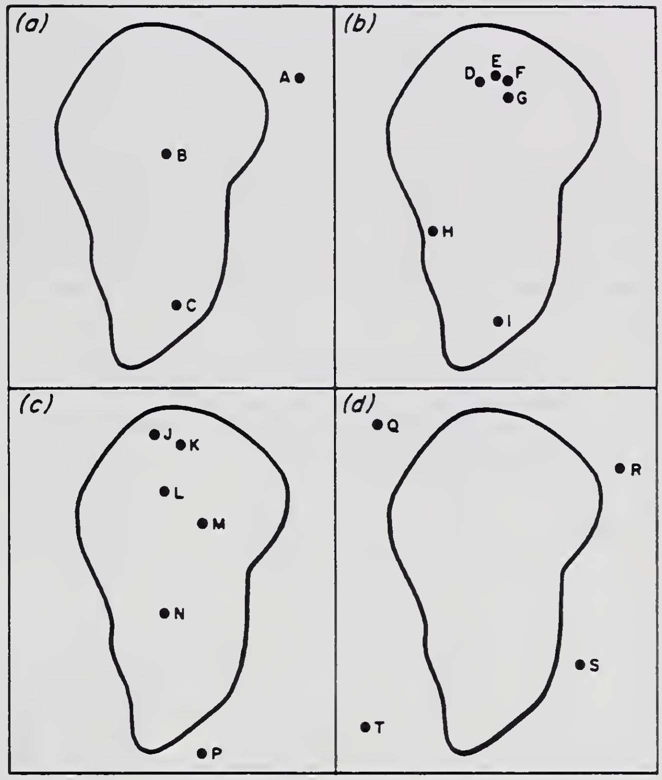
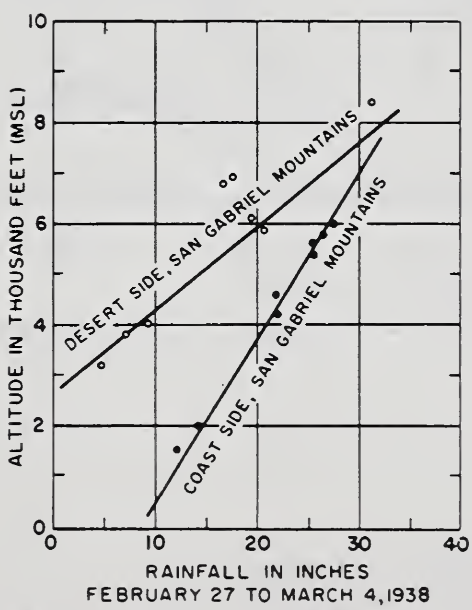


Figure 4.7.--Typical rain gage networks.



(FROM UNITED STATES GEOLOGICAL SURVEY- WATER SUPPLY PAPER NO. 844, FIGURE 5, PAGE 48.)

Figure 4.8.--Orographic influences. Points denote rain gage catches.

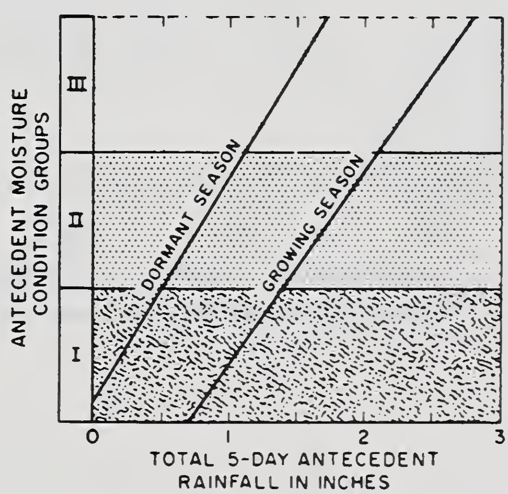


Figure 4.9.--Graph for estimating antecedent moisture condition.

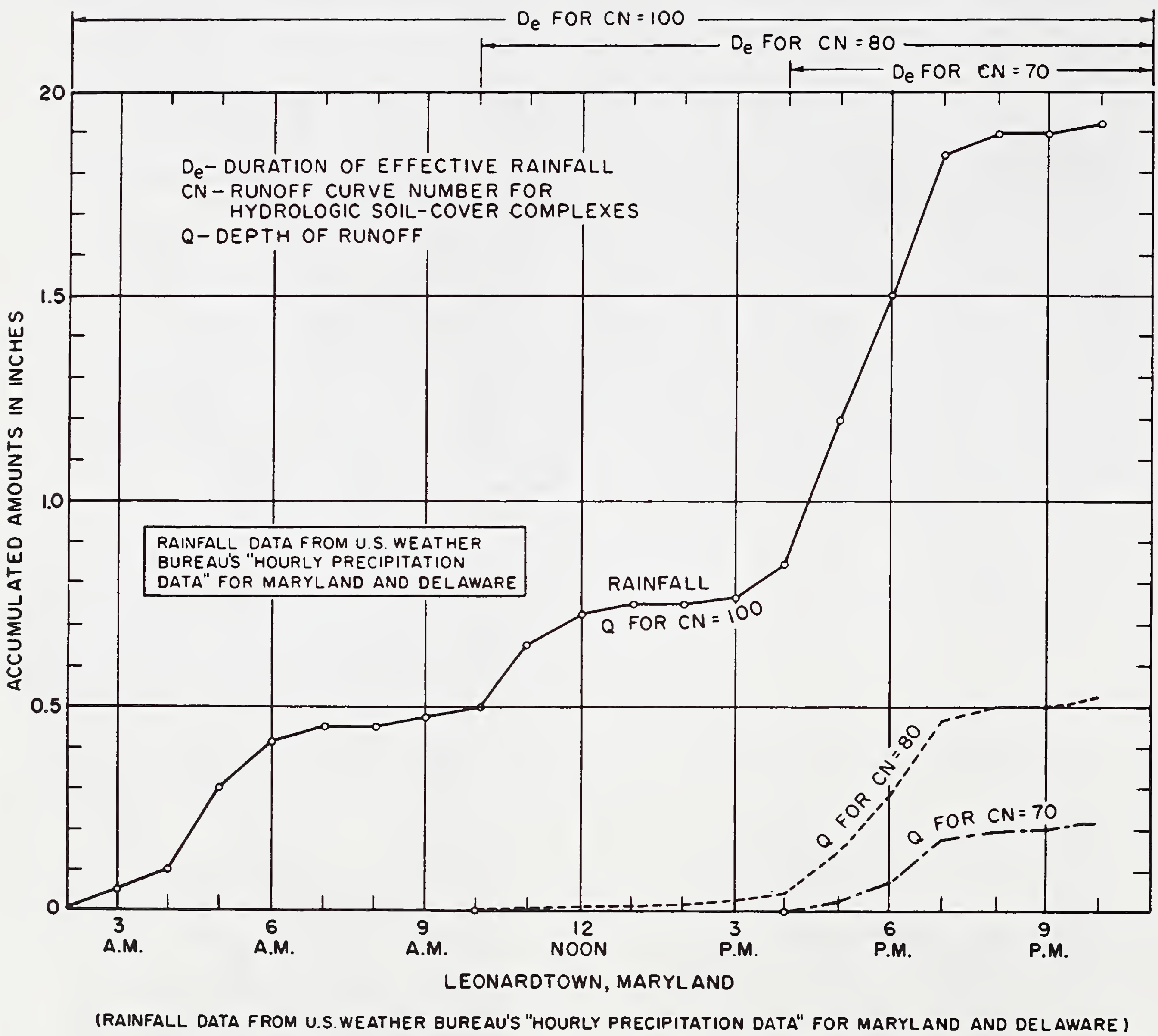


Figure 4.10.--Effect of hydrologic soil-cover complex on duration of effective rainfall.

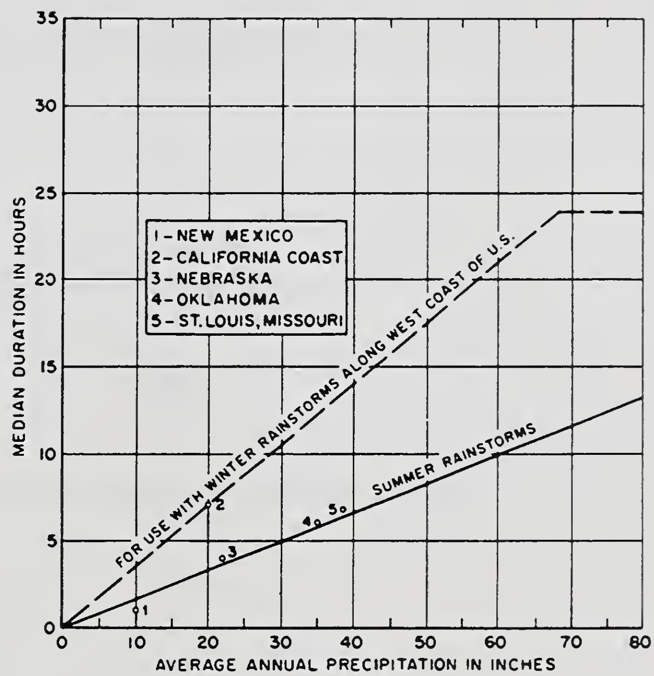


Figure 4.11.--Graph for estimating effective duration.

NATIONAL ENGINEERING HANDBOOK

SECTION 4

HYDROLOGY

CHAPTER 5. STREAMFLOW DATA

by

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SCS NATIONAL ENGINEERING HANDBOOK

SECTION 4

HYDROLOGY

CHAPTER 5--STREAMFLOW DATA

CONTENTS

U.S. Agricultural Research Service (ARS). The Soil and Water Conservation Research Division published its most recent compilation of small watershed data as "Hydrologic Data for Experimental Agricultural Watersheds in the United States, 1956-59," U.S. Dept. of Agric. Misc. Pub. 945, 674 pages, 1963. It is available from U.S. Government Printing Office, Washington, D. C. 20402, price $5.00. This book lists earlier publications of ARS compilations, which include data from watersheds formerly operated by SCS.

RELATED PUBLICATIONS. A list of streamflow stations having drainage areas of 400 square miles or less is given in "List of Selected Gaging Stations in the United States," by C. R. Gamble, U.S. Geological Survey, 91 pages, 1961. The listed stations are those for which USGS has compiled annual-flood and other data in SCS Projects 1 and 2. Copies of Gamble's report and of the project data were distributed to SCS State engineers and no additional copies are available. The project data will appear in a forthcoming WSP.

Descriptions of streamflow installations, methods of gaging, and other facts about USGS gaging practices are given in "Stream-Gaging Procedure," by Don M. Corbett and others, U.S. Geological Survey Water-Supply Paper 888, 245 pages, 1945; available from U.S. Government Printing Office, Washington, D. C. 20402, price $0.75. Similar information regarding Forest Service practices is in "Stream-Gaging Stations for Research on Small Watersheds," by Kenneth G. Reinhart and Robert S. Pierce, Agricultural Handbook 268, 37 pages, 1964; available from U.S. Government Printing Office, Washington, D. C. 20402, price $0.30. ARS practices are described in "Field Manual for Research in Agricultural Hydrology," Agricultural Handbook 224, 215 pages, 1962; available on request from U.S. Agricultural Research Service, Beltsville, Maryland 20705.

TEMPORARY STREAMFLOW-STATION INSTALLATIONS. The SCS cooperates with the USGS in the installation and operation of streamflow stations needed by SCS. This cooperation is on a formal administrative basis and the Chief, Hydrology Branch, can advise on the administrative procedure.

Sometimes a streamflow installation is needed for a brief period on a small stream, irrigation ditch, gully, or reservoir, and the circumstances do not justify the installation of a USGS station. If the flow to be measured is small, use can be made of measuring devices described in NEH-15:9, Measurement of Irrigation Water. If only the maximum stage or peak rate of flow is needed, a "crest staff gage" can be used at a culvert or other existing structure. Figure 5.1 shows a typical inexpensive staff gage. The pipe of the gage contains a

loose material (usually powdered cork) that floats and leaves a high-water mark or maximum stage. The stage is used with a rating curve (chap. 14) to estimate the peak rate of flow.

Some Uses of Streamflow Data

MEAN DAILY DISCHARGES

Records of mean daily discharges (or "mean dailies") are generally published in the form shown in figure 5.2, which is a typical page from a surface-water WSP. Summaries of discharge records appear in various forms; a typical page from a WSP containing summaries is shown in figure 5.3.

When using daily flow records it is often desirable to plot discharge against time in one of the two ways shown in figure 5.4. In a the mean dailies are plotted as point values at midday, with a logarithmic scale for discharge and an arithmetic scale for time. In b both scales are arithmetic. A plotting like a is used in studying low flows or recession curves and one like b in studying high flows or for showing discharges in their true proportions or for determining runoff amounts by measurement of areas. If a watershed has a lag of about 20 hours or more, mean dailies are suitable for plotting flood hydrographs because there is little chance that more than one peak occurs in any one day. But watersheds with shorter lags have flows that vary more widely during a day, so that a hydrograph of mean dailies may conceal important fluctuations; a continuous record of flow is used instead.

An important use of mean dailies is in computing storm runoff amounts including base flow (ex. 5.1) or excluding it (ex. 5.2).

Example 5.1.--Using data in figure 5.2, determine total runoff (including base flow) for the annual flood.

1. Determine the annual flood, the largest peak rate in the year. In figure 5.2 under "Extremes" read "Maximum discharge during year, 4,360 cfs Mar 4...."

2. Find the low point of mean daily discharge occurring before the rise of the annual flood. This point occurs on March 2 (table 5.1, an excerpt from figure 5.2).

Table 5.1.--Mean daily discharges, annual flood period

Date	Mean daily discharge	Remarks
	cfs	
Feb. 27	156	Flow from previous rise.
28	136	Same.
Mar. 1	126	Same.
2	105	Low point of flow.
3	* 222	Rise of annual flood begins.
4	* 3,630	Date of peak rate.
5	* 1,730	Flood receding.
6	* 558	Same.
7	* 320	Same.
8	* 191	Same.
9	* 146	End of flood period.
10	206	New rise begins

3. Find the date on the receding side of the flood when the flow is about equal to the low point of March 2. This second low occurs on March 9.

4. Add the mean daily discharges for the flood period from March 3 through March 9 (the starred discharges in table 5.1). The sum, which is the total runoff, is 6,797 cfs-days.

Runoff in cfs-days can be converted to another unit by use of an appropriate conversion factor (a table of factors follows chapter 22). For instance, to convert the result in example 5.1 to inches, use the conversion factor 0.03719, the sum of step 4, and the watershed drainage area in square miles (from fig. 5.2): 0.03719(6797)/258 = 0.9796 inches. Round to 0.98 inches.

If the flow on the receding side does not come down far enough, the usual practice is to make a "standard" recession curve out of well-defined recessions of several floods, fit this standard curve to the appropriate part of the plotted record, and estimate the mean dailies as far down as necessary.

If only the direct runoff (chap. 10) is needed, the base flow can be removed by any one of several methods. A simple method, accurate enough for most problems, is used in the next example.

Example 5.2.--Determine the direct runoff in inches for the annual flood of example 5.1.

1. Determine the total runoff in cfs-days (ex. 5.1).

2. Determine the average base flow for the flood period. This is an average of the flows on March 2 and March 9: (105 + 146)/2 = 125.5 cfs. Round to 126 cfs.

3. Compute the volume of base flow. Table 5.1 shows the flood period (starred discharges) to be 7 days; the volume of base flow is 7(126) = 882 cfs-days.

4. Subtract total base flow from total runoff to get total direct runoff: 6797 - 882 = 5915 cfs-days.

5. Convert to inches. Use the conversion factor 0.03719 (from the table following chapter 22), the total direct runoff in cfs-days from step 4, and the watershed drainage area in square miles (from the source of data, table 5.2): 0.03719(5915)/258 = 0.8527 inches. Round to 0.85 inches.

TRANSPOSITION OF STREAMFLOW RECORDS

Transposition of streamflow records is the use of records from a gaged watershed to represent the records of an ungaged watershed in the same climatic and physiographic region. Table 5.2 lists some of the kinds of data usually transposed and the factors affecting the correlations between data for the gaged and ungaged watersheds. The symbol A means that a considerable amount of analysis may be required before a transposition is justified.

Data are transposed with or without changes in magnitude, depending on the kind and the parameters influencing them. Runoff volumes of individual storms, for instance, are transposed without change in magnitude if the gaged and ungaged watersheds are alike in all respects. But if the hydrologic soil-cover complexes (CN) differ, it is necessary to use figure 10.1 as shown in the following example.

Example 5.3.--A gaged watershed with CN = 74 had a direct runoff of 1.60 inches. What is the comparable runoff for a nearby ungaged watershed with CN = 83?

1. Enter figure 10.1 with the runoff of 1.60 inches, go across to CN 74, go upward to CN 83, and at the runoff scale read a runoff of 2.29 inches.

Transposition of flood dates and number of floods per year is discussed in chapter 18; transposition of total and average annual runoff is discussed in chapter 20.

Table 5.2.--Factors affecting the correlation of data: a guide to the transposition of streamflow records

Kind of data	Factors:					
	Large distance between watersheds	Large difference in sizes of lag	Runoff from small-area thunder-storms	Large difference in sizes of drainage area	Snowmelt runoff on one only	Difference in hydrolog-ic soil-cover complexes
Flood dates	A	A	A	A	A	A
Number of floods per year	A	A	A	A	A	A
Individual flood, peak rate		A	A	A	A	A
Individual flood, volume			A	A	A	A
Total annual runoff			A	A	A	A
Average annual runoff				A	A	A

A means adverse effect on the correlation; if no A, the adverse effect is minor.

DETERMINATION OF HYDROLOGIC SOIL-COVER COMPLEX NUMBERS (CN)

Storm rainfall and streamflow data for annual floods are the best means by which CN can be established (chap. 9), and such CN are superior to those made by other means. The method of the following example is used; it applies only for antecedent moisture condition II (AMC-II), which is discussed in chapters 4, 9, and 10.

Example 5.4.--Given the rainfall and runoff data of columns 5 and 6 of figure 5.5 (a typical page from SCS Project 1), find the CN for AMC-II.

1. Fasten an overlay sheet over figure 10.1. The sheet must be transparent enough for the runoff curves to show through.

2. Plot runoff from column 5 against rainfall from column 6, as shown in figure 5.6(a).

3. Find which curve of figure 10.1 divides the plotting into two equal numbers of points. It may be necessary to interpolate between two curves; this can be done by penciling a curve on the overlay sheet. The CN for the selected curve is the CN for the watershed, in this example 65.

Figure 5.6(a) also shows the runoff curves for AMC-I and AMC-III (chap. 10). These were found by the relationship given in table 10.1, and no method comparable to that of example 5.4 is needed. But CN for specific antecedent conditions can be estimated by other methods, one of which is given in example 5.5 where antecedent base flow is used.

Example 5.5.--Use the data of figure 5.5 to determine the relation between antecedent base flow and the CN for a subsequent storm runoff.

1. Enter figure 10.1 with each storm rainfall (col. 6, fig. 5.5) and its runoff (col. 5) and find the CN for each storm.

2. Find the S value for each CN, using columns 1 and 4 of table 10.1.

3. Plot each antecedent base flow versus its associated S, using log paper as shown in figure 5.6(b), and draw the line of relation. Unless there is a strong indication of another slope, use a slope of -1 and locate the line so that an equal number of points falls on each side. Scales of CN on the margins make the graph easier to use.

Tests of the significance of such relationship and methods for using additional variables are discussed in chapter 18.

* * * *

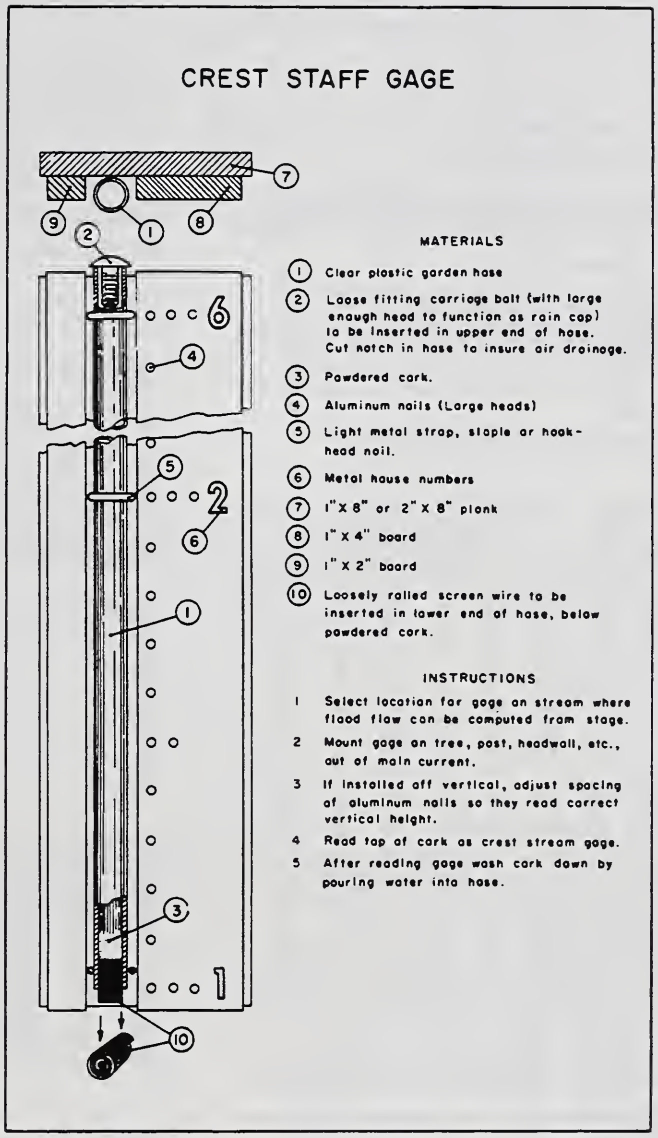

Figure 5.1.--Construction details of a crest staff gage.

WABASH RIVER BASIN 459

Wabash River near New Corydon, Ind.

Location.--Lat 40°33'50", long. 84°48'10", in SE¼ sec. 3, T. 24 N., R. 15 E., first principal meridian near center of span on downstream side of bridge on Indiana-Ohio State line road, 2 miles east of New Corydon and 2¼ miles downstream from Beaver Creek.

Drainage area.--258 sq mi.

Records available.--April 1951 to September 1953.

Gage.--Water-stage recorder. Datum of gage is 830.10 ft above mean sea level, datum of 1929. Prior to June 23, 1953, wire-weight gage at same site and datum.

Extremes.--Maximum discharge during year, 4,360 cfs Mar. 4 (gage height, 17.30 ft); minimum, 6.4 cfs Sept. 11, 28; minimum gage height, 5.75 ft Sept. 11.
1951-53: Maximum discharge, 4,690 cfs Mar. 11, 1952 (gage height, 17.59 ft); minimum, 1.3 cfs Aug. 18, 1951 (gage height, 5.40 ft).

Remarks.--Records good except those for periods of no gage-height record, which are fair.

Revisions.--WSP 1235: Drainage area.

Rating tables, water year 1952-53, except periods of ice effect (gage height, in feet, and discharge, in cubic feet per second)
(Shifting-control method used Oct. 1 to Nov. 23, Sept. 13-30)

Oct. 1 to Dec. 3

6.7	12	13.0	970
6.2	38	14.0	1,200
7.0	94	15.0	1,950
9.0	183	17.0	4,030
10.0	432		

Dec. 4 to Sept. 30

5.8	4.0	7.0	80
9.0	19	9.0	179
9.5	45	10.0	432

Note.--Same as preceding table above 10.0 ft.

Discharge, in cubic feet per second, water year October 1952 to September 1953

Day	Oct.	Nov.	Dec.	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.
1	19	15	a25	39	191	129	348	46	89	30	18	12
2	*20	14	*110	38	141	105	*248	40	93	28	30	12
3	19	14	1,340	b41	136	222	171	52	80	24	43	12
4	18	13	970	b45	131	3,630	156	29	77	21	30	13
5	18	16	600	b80	124	1,730	161	31	71	24	20	18
6	19	*14	240	b62	123	558	123	47	75	69	17	11
7	20	14	117	58	117	320	60	110	38	35	19	*10
8	20	18	82	79	104	191	46	284	37	22	38	9.9
9	18	19	78	271	*108	146	43	146	37	*19	20	8.9
10	17	a18	879	404	106	208	229	61	186	19	17	8.2
11	19	a19	800	770	240	358	117	43	914	19	19	9.9
12	24	19	480	358	684	285	79	55	208	17	15	11
13	20	19	108	252	348	302	80	44	90	15	16	11
14	19	19	79	629	208	432	64	34	59	19	14	11
15	18	18	87	814	179	915	58	97	48	19	14	13
16	19	19	64	419	158	572	371	124	40	19	14	10
17	18	17	50	252	148	382	176	1,120	38	19	13	9.9
18	18	18	40	1,080	b135	1,130	107	1,340	34	19	18	9.2
19	18	17	41	488	136	1,200	88	525	51	24	12	9.2
20	17	18	54	292	217	830	71	318	30	38	13	8.9
21	19	18	a100	272	1,850	446	81	119	28	28	12	9.8
22	18	29	a180	*188	572	579	54	487	24	19	12	9.9
23	18	58	a330	179	240	348	50	2,730	22	20	12	7.9
24	18	a130	*222	474	191	306	41	1,120	21	18	11	7.9
25	18	a300	128	480	178	306	42	800	22	18	11	7.8
26	16	a240	92	217	*168	292	48	379	21	13	12	8.2
27	18	a150	65	186	156	279	49	*198	20	14	12	7.8
28	18	a95	67	191	156	279	47	181	18	14	12	7.8
29	14	a85	45	158	-	265	43	125	18	15	12	7.8
30	15	a55	43	141	-	265	*46	105	20	*18	12	7.8
31	18	-	41	179	-	258	-	98	-	18	11	-
Total	528	1,431	7,294	8,760	9,901	19,938	3,256	10,765	2,183	661	514	256.7
Mean	17.0	47.7	235	283	248	548	109	347	73.1	21.5	19.9	8.89
Cfsm	-	-	-	-	-	-	-	-	-	-	-	-
In.	-	-	-	-	-	-	-	-	-	-	-	-

Calendar year 1952: Max 3,810 Min 18 Mean 271 Cfsm - In. -
Water year 1952-53: Max 3,630 Min 7.2 Mean 165 Cfsm - In. -

Peak discharge (base, 2,000 cfs).--Feb. 21 (9 a.m.) 2,330 cfs (15.40 ft) Mar. 4 (2 p.m.) 4,360 cfs (17.30 ft); Mar. 18 (7 p.m.) 2,330 cfs (15.38 ft); May 19 (5 a.m.) 2,030 cfs (15.10 ft); May 23 (8 a.m.) 3,260 cfs (19.26 ft)
* Discharge measurement made on this day.
a No gage-height record; discharge estimated on basis of 2 discharge measurements, weather records, and records for Wabash River at Bluffton.
b Stage-discharge relation affected by ice.

FIGURE 5.2 - A sample page from the U. S. Geological Survey's Surface Water-Supply Papers.

MUSKINGUM RIVER BASIN 269

243. Wakatomika Creek near Frazeysburg, Ohio

Location.--Lat 40°07'57", long. 82°08'53", in NW¼ sec. 13, T. 3 N., R. 9 W., 2 miles northwest of Frazeysburg, 2 miles downstream from Fivemile Run, and 2½ miles upstream from Black Run.

Drainage area.--140 sq mi.

Gage.--Water-stage recorder. Datum of gage is 748.12 ft above mean sea level, adjustment of 1912. Prior to Oct. 31, 1936, staff gage at same site and datum.

Average discharge.--14 years (1936-50), 158 cfs.

Extremes.--1936-50: Maximum discharge, 10,200 cfs Jan. 25, 1937 (gage height, 11.27 ft), from rating curve extended above 6,600 cfs on basis of velocity-area study; minimum, 3.1 cfs Aug. 12-14, 1944 (gage height, 1.20 ft).

Monthly and yearly mean discharge, in cubic feet per second

Water year	Oct.	Nov.	Dec.	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	The year
1936	-	-	-	-	-	-	-	-	-	-	-	14.3	-
1937	77.5	198	93.4	1,219	205	114	238	186	492	94.4	29.2	11.4	247
1938	22.4	30.9	136	121	305	441	464	163	70.4	23.0	108	63.3	191
1939	22.3	55.9	50.4	159	479	252	374	33.9	154	41.3	19.9	9.22	136
1940	25.1	17.3	32.9	36.6	238	427	654	163	222	105	92.5	46.1	172
1941	19.8	120	208	136	122	75.1	47.9	21.7	129	102	74.7	19.2	90.7
1942	45.8	107	105	80.5	271	225	234	108	127	35.5	19.1	12.9	113
1943	23.7	101	329	289	212	533	186	215	52.9	140	186	14.3	191
1944	14.7	20.9	15.9	22.9	94.2	460	422	102	62.9	9.68	22.0	19.7	103
1945	10.5	11.1	19.0	92.3	320	804	277	310	88.4	34.5	12.7	42.0	199
1946	113	173	143	110	329	260	70.4	219	270	49.7	49.2	9.47	148
1947	20.9	57.5	117	345	133	97.4	319	†430	514	108	79.9	79.9	179
1948	24.9	66.2	68.0	191	367	384	498	133	55.2	59.9	10.1	12.9	152
1949	22.7	58.9	122	491	349	298	199	129	38.4	199	42.9	38.9	191
1950	24.8	26.6	102	807	426	252	318	151	72.9	76.9	16.7	32.4	191

† Corrected.

Monthly and yearly runoff, in inches

Water year	Oct.	Nov.	Dec.	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	The year
1936	-	-	-	-	-	-	-	-	-	-	-	0.11	-
1937	0.64	1.57	0.77	10.04	1.52	0.94	1.90	1.33	3.92	0.79	0.24	.09	25.94
1938	.19	.24	1.50	1.00	2.25	3.93	3.99	1.19	.56	.19	.89	.50	15.91
1939	.18	.45	.42	1.31	3.54	2.08	2.98	.44	1.25	.34	.19	.05	13.19
1940	.21	.14	.27	.47	1.82	3.52	5.21	1.34	1.77	.86	.79	.37	19.74
1941	.14	.99	1.70	1.29	.91	.92	.38	.19	1.03	.94	.92	.15	8.91
1942	.38	.85	.86	.99	2.02	1.86	1.86	.89	1.01	.29	.13	.10	10.91
1943	.19	.80	2.70	2.38	1.57	4.39	1.49	1.75	.42	1.15	1.54	.11	18.48
1944	.12	.17	.13	.19	.50	3.79	3.38	.94	.50	.08	.18	.15	10.01
1945	.09	.09	.19	.51	2.38	9.92	2.21	2.33	.70	.29	.10	.33	19.02
1946	.93	1.38	1.19	.91	2.43	2.14	.38	1.80	2.18	.39	.40	.08	14.35
1947	.17	.49	.96	3.01	.99	.72	2.33	3.54	2.50	.89	.95	.64	17.08
1948	.20	.53	.56	1.33	2.83	3.19	3.97	1.29	.44	.53	.08	.10	14.79
1949	.19	.47	1.00	3.79	2.57	2.46	1.57	1.05	.47	1.40	.35	.31	15.93
1950	.20	.21	.84	6.64	3.17	2.08	2.53	1.24	.58	.63	.14	.26	18.52

Yearly discharge, in cubic feet per second

Year	W.S.P. no.	Water year ending Sept. 30: Momentary maximum: Discharge	Momentary maximum: Date	Minimum day	Mean	Per square mile	Runoff in inches	Calendar year: Mean	Calendar year: Runoff in inches
1936	823	-	-	-	-	-	-	-	-
1937	923	10,200	Jan. 25, 1937	9.9	247	1.79	25.94	234	22.88
1938	853	4,070	Apr. 7, 1938	9.2	191	1.15	15.91	154	14.94
1939	873	2,580	Jan. 30, 1939	3.2	136	.971	13.19	131	12.75
1940	883	9,940	Apr. 20, 1940	5.9	172	1.23	19.74	195	18.92
1941	923	1,480	July 19, 1941	9.9	90.7	.648	9.91	63.5	9.10
1942	953	2,220	Apr. 10, 1942	5.4	113	.807	10.91	129	12.51
1943	973	7,380	Mar. 20, 1943	7.0	191	1.38	18.48	157	15.21
1944	1003	4,900	Mar. 7, 1944	3.1	105	.738	10.01	102	9.95
1945	1033	9,040	Mar. 9, 1945	5.5	195	1.19	19.02	198	19.17
1946	1053	2,420	June 19, 1946	7.1	149	1.08	14.35	128	12.45
1947	1083	2,560	June 8, 1947	9.5	179	1.29	17.08	173	19.79
1948	1113	9,520	Feb. 14, 1948	5.1	152	1.09	14.79	138	15.19
1949	1143	2,700	Jan. 28, 1949	12	191	1.15	15.43	157	15.22
1950	1173	5,990	Jan. 6, 1950	8.9	191	1.38	18.52	-	-

FIGURE 5.3 - A sample page from U. S. Geological Water-Supply Paper 1305, "Compilation of Records of Surface Waters of the United States through September 1950."

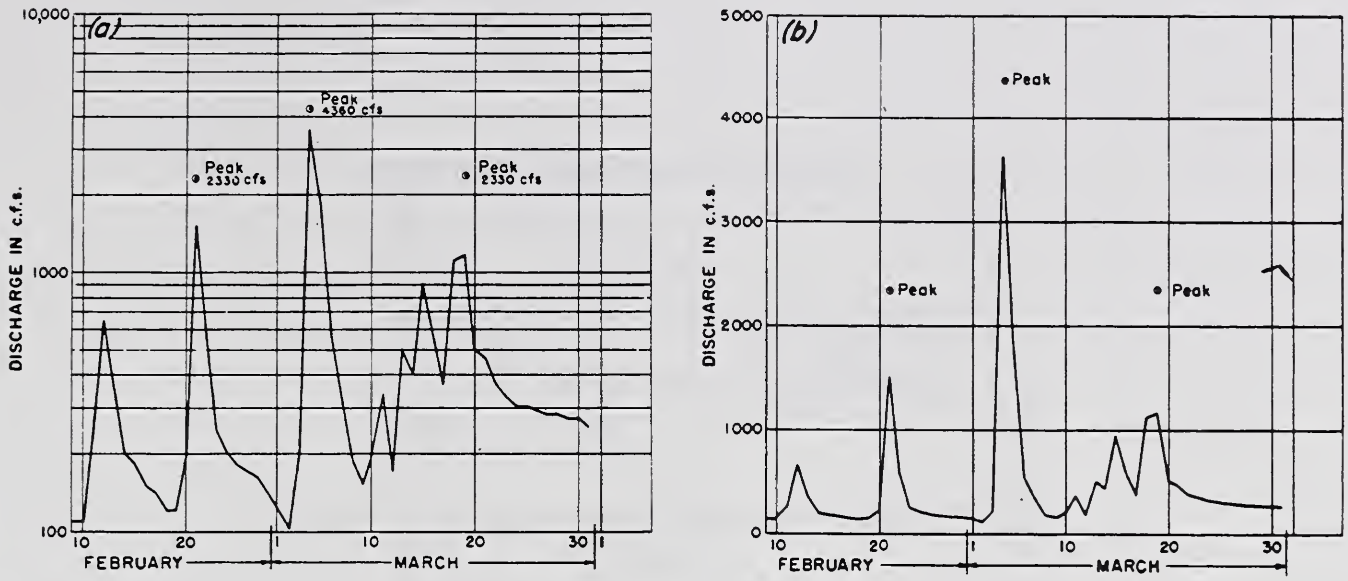

Figure 5.4.--Two methods of plotting daily flow records. In (a) the discharge scale is logarithmic; in (b) the scale is arithmetic.

14. Amicalola Creek near Dawsonville, Ga. (2B-3900) Lat 34°26', long 84°13'; drainage area, 84.7 sq mi; mean annual precipitation, 57.41.

Water year	Data relative to annual peak discharges								Annual runoff (in)
	Date	Peak discharge (cfs)	Antecedent base discharge (cfs)	Direct runoff (in)	Associated precipitation (in)	Storm duration (days)	Antecedent precipitation (in)		
							5-day	30-day	
1940	Aug. 13	2,500	81	0.81	4.99	1	0.30	2.67	22.37
1941	July 5	5,200	188	1.40	5.72	4	1.54	4.99	21.80
1942	Feb. 17	7,450	143	1.74	5.24	1	.20	4.93	30.94
1943	Dec. 29	2,680	232	1.65	4.31	2	1.36	6.63	39.12
1944	Mar. 19	3,460	305	1.16	3.80	2	.10	8.75	37.05
1945	Feb. 13	1,130	160	.36	1.95	1	.18	3.63	25.44
1946	Feb. 10	5,050	408	2.33	5.39	2	1.11	6.32	57.50
1947	Jan. 20	4,770	452	1.59	4.05	2	2.62	8.66	30.31
1948	Aug. 4	5,650	130	1.36	5.69	2	.40	8.46	34.98
1949	Nov. 28	5,500	204	1.85	5.59	3	1.48	9.53	48.93
1950	Mar. 13	3,460	276	1.15	3.77	2	1.22	4.74	37.91
1951	Mar. 29	2,380	189	1.33	4.71	2	.16	5.57	27.69
1952	Mar. 11	5,960	280	2.01	3.83	1	0	5.18	-

FIGURE 5.5 - A sample page from a compilation by the U.S. Geological Survey for SCS-Project No. 1.

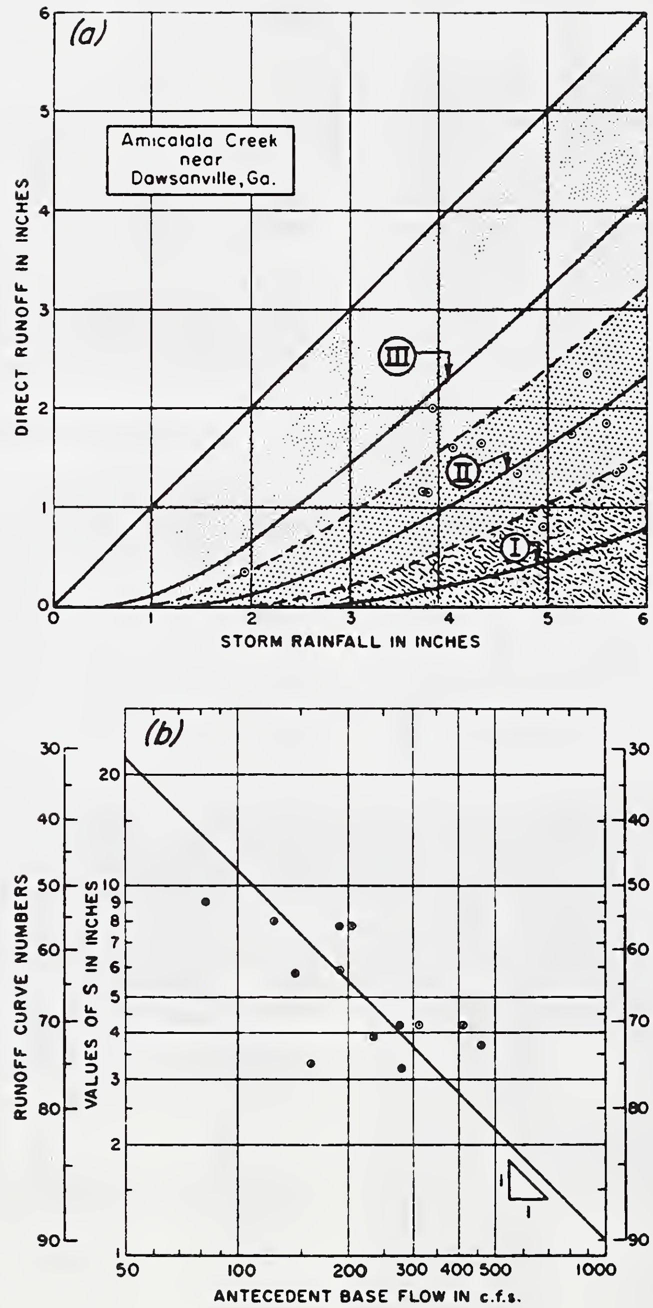

Figure 5.6.--Use of streamflow records for determination of (a) an average runoff curve number, and (b) a specific runoff curve number.

NATIONAL ENGINEERING HANDBOOK

SECTION 4

HYDROLOGY

CHAPTER 6. STREAM REACHES AND HYDROLOGIC UNITS

by

Victor Mockus
Hydraulic Engineer

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SCS NATIONAL ENGINEERING HANDBOOK

SECTION 4

HYDROLOGY

CHAPTER 6--STREAM REACHES AND HYDROLOGIC UNITS

CONTENTS

CHAPTER 6. STREAM REACHES AND HYDROLOGIC UNITS

The stream system of a watershed is divided into reaches, and the watershed into hydrologic units, for the convenience of work during project formulation. This chapter gives some details on the selection of reaches for hydrologic or economic studies, presents alternative means for studies of alluvial fans, and briefly describes a hydrologic unit and its use in a project study.

Reaches

A reach is a length of stream or valley used as a unit of study in project formulation. It contains a specified feature that is either fairly uniform throughout (as hydraulic characteristics or flood damages) or requires special attention in the study (as a bridge). Reaches are shorter for hydraulic than for economic studies so that it is best to consider hydraulic needs first when selecting reaches, afterward combining the hydraulic reaches into longer ones for the economic study.

Reaches are physically defined at each end by cross sections that usually extend across the valley. A cross section is either straight and at a right angle to the major path of flow in the valley, or it is a connected series of segments that are at right angles to flows in their vicinity. The "head" and "foot" of a reach are the upstream and downstream ends respectively. "Right bank" and "left bank" are designated looking downstream. For reference, reaches and cross sections are numbered in any simple and consistent way such as the one in figure 6.1 and table 6.1. But if an electronic computer program (chap. 2) will be used, the numbering must follow the system specified in the program.

The purpose of a reach determines which relationships of the reach must be developed from field surveys. For a hydrologic study the required relationships include those of stage and discharge (chap. 14), stage and end-area (chaps. 14 and 17), and, if manual flood routings will be made, discharge and velocity (chap. 14). For an economic study they are stage and discharge (chap. 14), stage and area-inundated (chap. 13), and stage and damage (Economics Guide, chap. 3).

Table 6.1. Reach and cross-section data

Reach No. 1/	Cross-section No.	Cross-section stationing	Length of reach 2/	Travel time 3/	Accumulated drainage area	Soil-cover complex No. 4/ Present	Soil-cover complex No. 4/ Future
			Feet	Hours	Square miles		
4			7500	0.60		80	78
	FR-1	2231 + 00			3.6 5/		
	BB	2192 + 00			4.0 6/		
	AA	2160 + 00			4.4 7/		
6			15600	1.50		80	78
	FF	2138 + 00			7.5 5/		
	EE	2100 + 00			8.0		
	DD	2054 + 00			8.4		
	CC	2016 + 00			8.8		
	BB	2014 + 00			8.8		
	AA	2012 + 00			8.9 7/		

1/ Reach No. is same as subdivision No.
2/ Channel length of reach.
3/ Travel time of a 2-year frequency flow through the reach
4/ Soil-cover complex Nos. for the total area above the foot of the reach. They were obtained by weighting (chap. 10).
5/ Drainage area at the head of the reach.
6/ The drainage area at this cross section was estimated.
7/ Drainage area at the foot of the reach.

LOCATION

The head or foot of a reach is at or near one of the following places on a stream:

1. Boundary of an agricultural area having flood damages.

2. Boundary where agricultural damages change significantly.

3. Boundary of an urban area, oil-storage field, or any other area of high potential flood damage for which levees or other local protective works may be proposed.

4. Junction of a major tributary and the main stream.

5. Station where streamflow is gaged.

6. Installation controlling streamflow, such as a weir or a culvert in a high road fill.

7. Installation restricting streamflow, such as a bridge.

8. Site proposed for a floodwater-retarding or other structure.

9. Section where shape or hydraulic characteristics of the channel or valley change greatly.

10. Section where channel control creates large storage upstream.

In selecting reaches it must be kept in mind that the method of computing water-surface profiles may specify a maximum permissible length of reach. Some electronic-computer programs have a built-in routine for transposing or interspersing auxiliary cross sections to avoid stopping the machine when an excessive length of reach is encountered in the data. Even these programs have limitations that must be observed.

Locations for reaches are selected by the hydrologist and others in the work plan party. Tentative locations are made during the preliminary investigation of a watershed (chap. 3) and shown on a base map or aerial photograph. Low-altitude aerial reconnaissance may be necessary for locating reaches in watersheds without access roads or where timber, brush, or cultivated crops obstruct vision at the ground level. If flood damage studies will be made, flood-plain areas with potentially high damage are also located and shown. The map or photograph is later used for identifying the reaches that need most attention in the studies. Once the relative

importance of the reaches is known, the hydrologist selects the locations of cross sections and determines the intensities of work to be done by the field survey crew.

MEASUREMENT

The measurements made during a field survey are usually those necessary to define the changes in ground elevation in the line of a cross section and the horizontal distances between sections. At this same time it is necessary to estimate Manning's n for hydraulic computations (chap. 14). The value of n must represent roughness conditions for the full length of the reach. If a cross section is divided into segments, the n for each segment applies to a strip through the reach.

Length

The length of a reach is the distance between cross sections at the head and foot, measured along the sinuous path of flow in the channel or valley. The channel is nearly always longer than the valley so that two lengths may be applied in a study: the channel length when the flow is low (within banks of the channel) and the valley length when the flow is over the flood plain. This means that as a flood rises the reach becomes shorter, a change that must be taken into account when computing water-surface profiles (chap. 14) and flood damages (chap. 13). Reach lengths are generally determined by use of an aerial photograph or a detailed topographic map because the paths of flow are often complex and not easy to determine in the field.

Profile

Elevations of cross sections are related to a common datum if profiles of the valley or channel are needed for computation of water-surface profiles by the Leach, Escoffier, or Doubt methods (chap. 14).

Hydraulic Roughness

Estimates of hydraulic roughness (Manning's n) are made by the procedure given in NEH-5, Supplement B, or an equivalent procedure. Chapter 14 contains a discussion of Manning's n and its variations in natural channels.

REACH DATA FOR A COMPUTER PROGRAM

If water-surface profile or similar computations will be made by an electronic computer, the computer-program description should be examined for limitations on the input data such as length of reach and number of elements in a cross section. These limitations must be kept in mind when working instructions are given to the survey crew. Typical limitations are given in SCS Technical Release 14, "Computations of Water Surface Profiles and Related Parameters by the IBM 650 Computer."

Alluvial Fans

Alluvial fans, also called debris slopes or debris fans, are sediment deposits formed where the grade of a mountain stream is abruptly reduced as the stream enters an area of gentler slope, such as the valley of another stream. Large fans may be inhabited or have agricultural use. The paths of flood flows shift from one side to another of a fan so that reaches are useless and a special method for project evaluation must be adopted. In this method the floodwater damages on alluvial fans are related to actual or estimated runoff volumes that are referenced to an upstream cross section above the fan, such as a stream gage or other control section. The evaluation of flood damages follows this order:

1. Information about the monetary value of damages for each known flood on the fan is obtained by interviews or from historical sources.

2. The volume of flood runoff for each flood is determined from streamflow records or estimated by use of rainfall and watershed data and the methods shown in chapter 10.

3. The relation between flood runoffs and damages is developed (see the Economics Guide).

4. The frequencies of flood-runoff amounts are estimated (chap. 18).

5. A damage-frequency curve is developed (chap. 18).

6. The average annual damage is determined (chap. 18).

7. The effects of a proposed upstream project on the amounts of runoff are determined. The amounts (and therefore the flood

damages) decrease when changes in land use and treatment decrease the hydrologic soil-cover complex number (chap. 10) or when storage structures reduce flood flows (chap. 17).

8. The runoff-damage relation of step 3 is used with the reduced runoffs of step 7 to estimate damages still remaining.

9. A modified damage-frequency curve is developed and plotted on the graph used in step 5.

10. The difference between present and future damage-frequency curves is obtained as shown in chapter 18 to estimate the project benefits.

Hydrologic Units

When a large watershed or a river basin is studied it is desirable to divide the watershed or basin into subareas or subwatersheds called hydrologic units (HU) and to make the study in terms of these units. By so doing, the work is more easily handled and the study takes less time. An HU may also be used as a sample watershed; that is, project costs and benefits within a selected HU are evaluated in detail and afterward applied to other similar HU's for which no internal evaluation is made. The small watershed in figure 6.1 has enough detail for a sample watershed.

Each HU is the drainage area of a minor tributary flowing into the main stream or a major tributary. Areas between minor tributaries are combined and also used as HU's. Cross sections and reaches are needed only when an HU is a sample watershed. Storms in the historical or frequency series (chap. 18) are developed on an HU basis, as are runoff curve-numbers and hydrographs. Hydrographs for present and with land use and treatment conditions are developed for an entire HU with reference to the HU outlet (chap. 16).

If an HU contains structural measures that affect the rates of a hydrograph, the changes are determined by short-cut methods of routing (chap. 17) and the modified hydrograph, like the others, is referenced to the HU outlet. The watershed or basin flood routing is carried out on the major tributaries and main stream, with the HU's supplying the starting and local inflow hydrographs.

* * * *

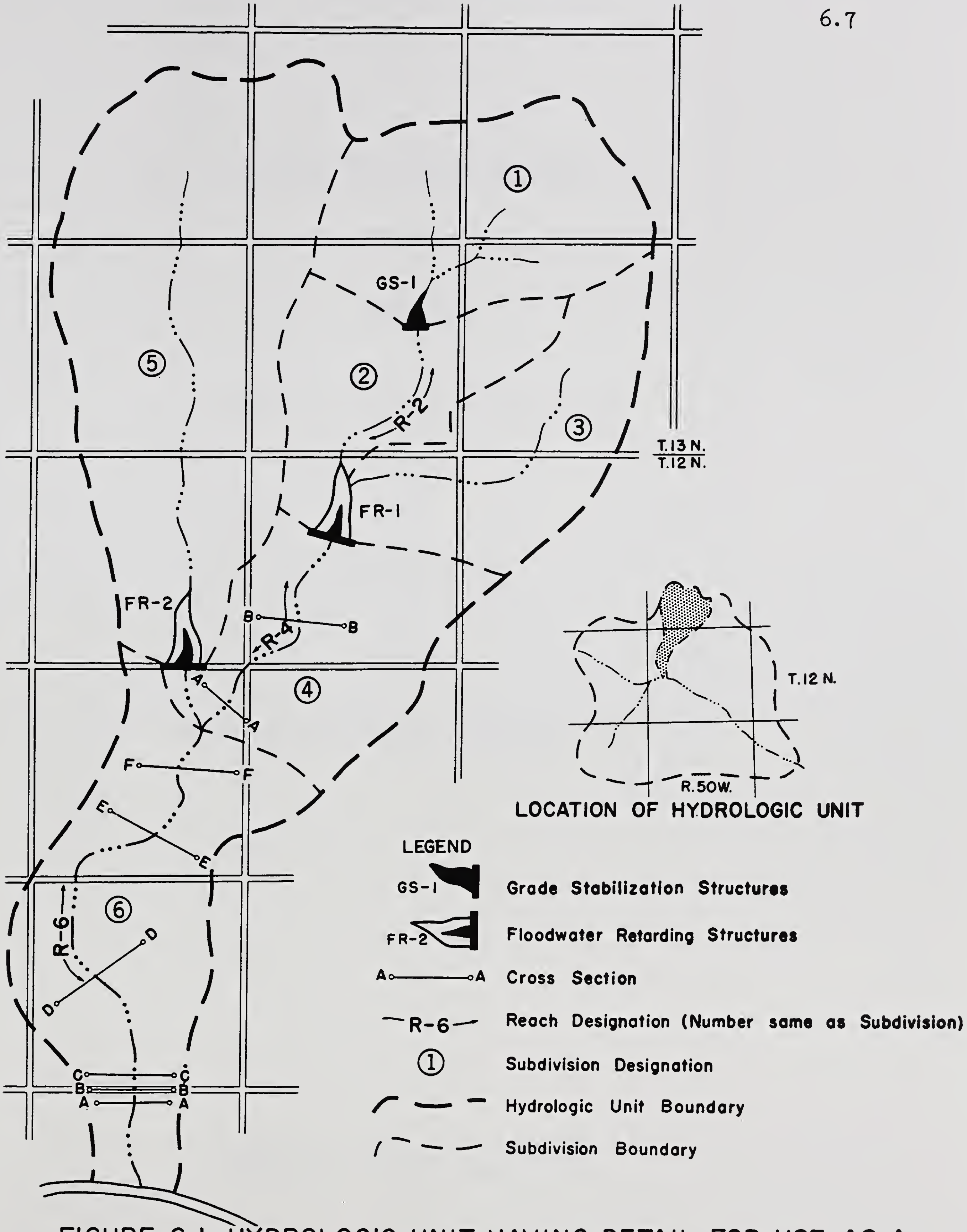

FIGURE 6.1—HYDROLOGIC UNIT HAVING DETAIL FOR USE AS A SAMPLE WATERSHED

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HYDROLOGY

CHAPTER 7. HYDROLOGIC SOIL GROUPS

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SECTION 4

HYDROLOGY

CHAPTER 7--HYDROLOGIC SOIL GROUP

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CHAPTER 7. HYDROLOGIC SOIL GROUPS

This chapter gives definitions of four soil groups that are used in determining hydrologic soil-cover complexes (chap. 9), which are used in a method for estimating runoff from rainfall (chap. 10). A table gives the group-classifications of more than 4,000 soils in the United States and Puerto Rico. Methods of making and using the classifications are briefly discussed.

Watershed-Soils Classification

Soil properties influence the process of generation of runoff from rainfall and they must be considered, even if only indirectly, in methods of runoff estimation. When runoff from individual storms is the major concern, as in flood prevention work, the properties can be represented by a hydrologic parameter: the minimum rate of infiltration obtained for a bare soil after prolonged wetting. The influences of both the surface and the horizons of a soil are thereby included. The influence of ground cover is treated independently, as discussed in chapters 8, 9, and 10.

The parameter, which indicates the runoff potential of a soil, is the qualitative basis of the classification in this chapter of all soils into four groups. The classification is broad but the groups can be divided into subgroups, as shown in example 7.1, whenever such a refinement is justified. Chapter 9 describes how the groups are given quantitative significance in the runoff-estimation method of chapter 10.

DEFINITIONS

In the definitions to follow, the infiltration rate is the rate at which water enters the soil at the surface and which is controlled by surface conditions, and the transmission rate is the rate at which the water moves in the soil and which is controlled by the horizons. The hydrologic soil groups, as defined by SCS soil scientists, are:

A. (Low runoff potential). Soils having high infiltration rates even when thoroughly wetted and consisting chiefly of deep, well to excessively drained sands or gravels. These soils have a high rate of water transmission.

B. Soils having moderate infiltration rates when thoroughly wetted and consisting chiefly of moderately deep to deep, moderately well to well drained soils with moderately fine to moderately coarse textures. These soils have a moderate rate of water transmission.

C. Soils having slow infiltration rates when thoroughly wetted and consisting chiefly of soils with a layer that impedes downward movement of water, or soils with moderately fine to fine texture. These soils have a slow rate of water transmission.

D. (High runoff potential). Soils having very slow infiltration rates when thoroughly wetted and consisting chiefly of clay soils with a high swelling potential, soils with a permanent high water table, soils with a claypan or clay layer at or near the surface, and shallow soils over nearly impervious material. These soils have a very slow rate of water transmission.

The Soil List

The list at the end of this chapter contains the names of more than 4,000 soils in the United States and Puerto Rico. The capital letter following a name designates the hydrologic soil group classification.

The original classifications were based on the use of rainfall-runoff data from small watersheds or infiltrometer plots, but the majority are based on the judgments of soil scientists and correlators who used physical properties of the soil in making their decisions. They classified a soil in a particular group by comparing its profile with profiles of soils already classified. They assumed that the soil surfaces were bare, maximum swelling had taken place, and rainfall rates exceeded surface intake rates. Thus, most of the classifications are based on the premise that similar soils (similar in depth, organic-matter content, structure, and degree of swelling when saturated) will respond in an essentially similar manner during a rainstorm having excessive intensities.

The classification of a soil in the list can be checked by using the procedure of example 5.4. The soil in question must be the only one

on the watershed and rainfall-runoff data for bare-soil periods must be available. Checks that have been made so far have not caused any changes in the present classification.

USE OF THE SOIL GROUPS

To use the soil list it is necessary to know only the names of the soils on the watershed being studied. To use the classification in estimating runoff (chap. 10) it is also necessary to know the area of each soil and, if the watershed is large, its location by hydrologic units (chap. 6). The SCS hydrologist usually consults a State soil scientist when soils of a watershed are to be classified. If there is no soil survey for the watershed the consultant can usually get adequate information from work unit personnel. Making a soil survey solely for hydrologic purposes is seldom justifiable. It should take less than a day to classify the soils on a 400-square-mile watershed. Often, when working with a watershed in familiar territory, the hydrologist needs little more than a check on his own estimates of the groupings.

Determining Areal Extents

Precise measurement of soil-group areas, such as by planimetering soil areas on maps or weighing map cuttings, is seldom necessary for hydrologic purposes. The maximum detail should not go beyond that illustrated in figure 7.1: in a the individual soils in a hydrologic unit are shown on a sketch map; in b the soils are classified into groups; in c a grid (or "dot counter") is placed over the map and the number of grid intersections falling on each group is counted and tabulated; in d is shown the tabulation and a typical computation of a group percentage. Simplified versions of this procedure are generally used in practice.

Number of Soil Groups to be Used

Often one or two soil groups predominate in a watershed, others covering only a small part. Whether the small groups should be combined with the predominate ones depends on their classifications. For example, a hydrologic unit with 90 percent of its soils in the A group and 10 percent in the D will have most of its storm runoff coming from the D soils and putting all soils into the A groups will cause a serious under-estimation of runoff. If the groups are more nearly alike (A and B, B and C, or C and D) the under- or over-estimation may not be as serious but a test may be necessary to show this. Rather than test each case, follow the

rule that two groups are combined only if one of them covers less than about 3 percent of the hydrologic unit. Impervious surfaces should always be handled separately because they produce runoff even if there is none from D soils.

Subgroups

If subgroups are used, the runoff curve numbers (CN) for them can be determined by linear interpolation on table 9.1 or, more elaborately, by the method of the following example.

Example 7.1.--A soil is to be classified in a subgroup falling midway between groups B and C. The land uses are "Row crops, straight-row, good rotation" and "Legumes, straight-row, good rotation" (see table 9.1). Determine the CN for the subgroup.

1. Use table 9.1 to find the CN for each of the four soil groups and two land uses. The results are:

Land uses	Soil groups			
	A	B	C	D
Row crops, straight-row, good rotation	67	78	86	89
Legumes, straight-row, good rotation	58	72	81	85

2. Plot the four CN for each land use as shown in figure 7.2, using each CN as the midpoint of a soil group, and draw a curve through the points. Each land use has its own curve.

3. Interpolate on the group scale and find the CN for each land use. For this example the subgroup is midway between the B and C groups so that the CN is 82 for the row crop and 77 for the legume. Linear interpolation on table 9.1 gives 81.5 and 76.5, respectively, which are rounded to 82 and 76.

The subgroup in example 7.1 can be designated the B- or C+ subgroup. More elaborate classifications (B_1, B_2, B_3, etc.) are not justified unless the soil classifications were made using rainfall-runoff data.

Reclassification of a Soil

Some of the soils in table 7.1 are in the D group because of a high water table that creates a drainage problem. Once these soils are

effectively drained they can be placed in an alphabetically higher group. They can be classified locally on a case by case basis.

When there is a need to reclassify a soil on the basis of additional data, the SCS State soil scientist should submit the case for consideration to the soil correlator for that area.

* * * *

TABLE 7.1--HYDROLOGIC GROUPS OF THE SOILS OF THE UNITED STATES

AABAB	D	ADKINS, HARDPAN	B	ALBEE	C	ALPOWA	B	ANDERLY	C
AABERG	D	SUBSTRATUM		ALBEMARLE	B	ALROS	C	ANDERS	C
AASTAD	B	ADKINS, GRAVELLY	B	ALBERTVILLE	C	ALSCO	B	ANDERSON	B
AAZDAHL	B	SUBSTRATUM		ALBINAS	B	ALSPAUGH	C	ANDOK	B
ABAC	D	ADLER	C	ALBION	B	ALSTAD	C	ANDOVER	D
ABAJO	C	ADNAN	D	ALBRIGHTS	C	ALSTONY	B	ANDRADA	D
ABARCA	B	ADOLPH	B/D	ALBUS	B	ALSUP	C	ANDREESON	C
ABBOTT	D	ADOS	C	ALCESTER	B	ALTAMONT	D	ANDREGG	B
ABBOTTSTOWN	C	ADRIAN	A/D	ALCOA	B	ALTAVISTA	C	ANDRES	B
ABCAL	D	ADVOKAY	D	ALCONA	B	ALTDORF	D	ANDREWS	C
ABEGG	B	AECET	C	ALCORN	B	ALTHOUSE	B	ANDRY	D
ABELA	B	AENEAS	B	ALCOT	A	ALTICREST	B	ANEO	D
ABELL	B	AFTADEN	D	ALCOVA	B	ALTMAR	B	ANETH	B
ABERDEEN	C	AFTON	C/D	ALDA	C	ALTO	C	ANGELICA	B/D
ABERONE	B	AGAIPAH	D	ALDA, SALINE	B/D	ALTOGA	C	ANGELINA	D
ABERT	B	AGAN	D	ALDA, CHANNELED	C	ALTON	A	ANGELO	C
ABES	D	AGAR	B	ALDAX	D	ALTOONA	C	ANGELUS	B
ABGESE	B	AGASSIZ	D	ALDEN	D	ALTURAS	C	ANGIE	D
ABILENE	C	AGATE	D	ALDER	C	ALTUS	B	ANGLE	A
ABIQUA	C	AGATHA	B	ALDERMAND	B	ALTVAN	B	ANGLEN	C
ABO	C	AGAWAM	B	ALDERWOOD	C	ALUF	A	ANGOLA	C
ABOR	D	AGENCY	C	ALDI	D	ALUN	B	ANGORA	B
ABRA	C	AGER	D	ALDINE	D	ALUSA	D	ANGOSTURA	B
ABRA, BEDROCK	B	AGET	B	ALDING	D	ALVARADO	B	ANHALT	D
SUBSTRATUM		AGNAL	D	ALDINO	C	ALVIN	B	ANIAK	D
ABRA, DRY	B	AGNESTON	B	ALEDO	C	ALVIRA	C	ANIMAS	C
ABRAHAN	B	AGNESTON, COBBLY	C	ALEGROS	C	ALVISO	D	ANINTO	D
ABRAZO	D	SUBSTRATUM		ALEKNAGIK	C	ALVOR	D	ANITA	D
ABRAZO, GRAVELLY	C	AGNEW	C	ALEMEDA	C	ALVOR, DRAINED	C	ANKENY	B
ABRAZO, COBBLY	D	AGNOS	D	ALEX	B	ALWILDA	B	ANKLAM	D
ABREU	B	AGUA	B	ALEXANDER	C	ALZADA	D	ANKONA	D
ABSAROKEE	C	AGUA DULCE	B	ALEXANDRIA	C	ALZOLA	D	ANNABELLA	B
ABSCOTA	A	AGUA FRIA	B	ALFIR	B	AMADOR	D	ANNANDALE	C
ABSHER	D	AGUADILLA	A	ALFORD	B	AMAGON	D	ANNAW	B
ABSTED	C	AGUALT	B	ALGANSEE	B	AMALIA	B	ANNEMAINE	C
ABSTON	C	AGUEDA	B	ALGARROBO	A	AMALU	D	ANNIS	C
ACACIO	B	AGUILARES	B	ALGERITA	B	AMANA	B	ANNIS, SALINE	B
ACADENY	C	AGUILITA	B	ALGIERS	C/D	AMANDA	C	ANNIS,	C
ACADIA	D	AGUIRRE	D	ALGOA	C	AMARILLO	B	SALINE-ALKALI	
ACANA	D	AGUSTIN	B	ALGOMA	B/D	AMASA	B	ANNIS, DRAINED	B
ACANOD	C	AHL	C	ALHAMBRA	B	AMBER	B	ANNISQUAM	C
ACASCO	D	AHLSTROM	D	ALHARK	B	AMBIA	D	ANNISTON	B
ACEITUNAS	B	AHMEEK	C	ALICE	B	AMBOAT	C	ANNONA	D
ACEL	C	AHOLT	D	ALICEL	B	AMBOY	C	ANOCON	C
ACKER	B	AHREN	B	ALICIA	B	AMBRANT	B	ANOKA	B
ACKERNAN	A/D	AHRNKLIN	C	ALIDA	B	AMBRAW	B/D	ANONES	C
ACKERVILLE	C	AHTANUM	D	ALIKCHI	B	AMELIA	B	ANSARI	D
ACKETT	D	AHTANUM, DRAINED	C	ALINE	A	AMENIA	B	ANSEL	B
ACKLEY	B	AHWAHNEE	B	ALKO	D	AMENSON	D	ANSELMO	B
ACKMEN	B	AIBONITO	C	ALLAGASH	B	AMERICUS	A	ANSELMO, BEDROCK	A
ACKMORE	B	AIDO	D	ALLAMORE	D	AMERY	B	SUBSTRATUM	
ACKWATER	D	AIKEN	B	ALLANTON	B/D	AMES	C/D	ANSGAR	B/D
ACME	C	AIKMAN	D	ALLARD	B	AMESHA	B	ANSPING	B
ACO	B	AIKMAN, STONY	C	ALLEGHENY	B	AMESMONT	C	ANT FLAT	C
ACOMA	C	AILEY	B	ALLEMANDS	D	AMHERST	D	ANTEL	C
ACORD	C	AIMELIIK	B	ALLEN	B	AMISTAD	D	ANTELOPE SPRINGS	C
ACOVE	C	AINAKEA	B	ALLENDALE	B	AMITY	D	ANTERO	C
ACREDALE	D	AINSLEY	B	ALLENS PARK	B	AMMON	B	ANTHO	B
ACREE	C	AINSWORTH	B	ALLENTINE	D	AMOLE	A	ANTHONY	B
ACRELANE	C	AIRPORT	D	ALLENWOOD	B	AMOR	B	ANTIGO	B
ACTON	B	AITS	B	ALLEY	B	AMOS	C	ANTILON	B
ACUFF	B	AJO	C	ALLHANDS	D	AMOSTOWN	C	ANTIOCH	D
ACUNA	C	AKAKA	A	ALLIANCE	B	AMPAD	C	ANTLER	C
ACY	C	AKAN	B/D	ALLIGATOR	D	AMPHION	C	ANTOINE	B
ADA	C	AKASKA	B	ALLIS	D	AMSDEN	B	ANTONITO	C
ADAIR	C	AKELA	D	ALLISON	B	AMSTERDAM	B	ANTOSA	D
ADAMS	A	AKERCAN	B	ALLITRAL	B	AMTOFT	D	ANTROBUS	B
ADAMSON	B	AKERUE	D	ALLOR	B	AMWELL	C	ANTWERP	C
ADAMSVILLE	C	AKLER	D	ALLOUEZ	B	AMY	D	ANTY	B
ADATON	D	ALADDIN	B	ALMAC	B	ANACAPA	B	ANUNDE	B
ADAVEN	C	ALADSHI	B	ALMAVILLE	D	ANACOCO	D	ANVIK	B
ADDICKS	D	ALAE	A	ALMENA	C	ANACONDA	B	ANWAY	B
ADDIELOU	B	ALAELOA	B	ALMIRANTE	B	ANAHEIM	C	AOWA	B
ADE	A	ALAGA	A	ALMO	D	ANAHUAC	D	APACHE	D
ADEL	B	ALAKAI	D	ALMONT	D	ANAMITE	D	APAKUIE	A
ADELAIDE	D	ALAMA	B	ALMOTA	C	ANAPRA	B	APALACHEE	D
ADELANTO	B	ALAMANCE	B	ALMY	B	ANASAZI	C	APALO	B
ADELINO	B	ALAMO	D	ALNITE	D	ANASAZI, NONSTONY	B	APELDOORN	A
ADELINO,	C	ALAMOGORDO	B	ALO	D	ANASAZI, DRY	B	APISHAPA	C
SALINE-ALKALI		ALAMOSA	D	ALOHA	C	ANATONE	D	APISON	B
ADELPHIA	C	ALAMOS	B	ALOMAX	D	ANAVERDE	B	APMAT	B
ADENA	C	ALAPAHA	D	ALONA	B	ANAWALT	D	APMAY	D
ADGER	D	ALAPAI	A	ALONSO	B	ANCHO	B	APOLLO	B
ADILIS	B	ALAZAN	B	ALOVAR	C	ANCHOR POINT	D	APOPKA	A
ADJUNTAS	C	ALBAN	B	ALPENA	A	ANCHORAGE	A	APPANOOSE	D
ADKINS	B	ALBANO	D	ALPHA	B	ANCLOTE	D	APPERSON	C
ADKINS, ALKALI	B	ALBANY	C	ALPIN	A	ANCO	C	APPIAN	B
ADKINS, WET	C	ALBATON	D	ALPON	B	ANDERGEORGE	B		

NOTES: TWO HYDROLOGIC SOIL GROUPS SUCH AS B/C INDICATES THE DRAINED/UNDRAINED SITUATION.
MODIFIERS SHOWN, E.G., BEDROCK SUBSTRATUN, REFER TO A SPECIFIC SOIL SERIES PHASE FOUND IN SOIL MAP LEGEND.

TABLE 7.1--HYDROLOGIC GROUPS OF THE SOILS OF THE UNITED STATES

APPIAN,	B	ARMAGH	D	ATCO	B	AZTEC	C	BARABOO	B
SALINE-ALKALI		ARMCO	C	ATENCIO	B	AZULE	C	BARAGA	C
APPIAN, WET	C	ARMELLS	B	ATEPIC	D	AZWELL	B	BARATARI	A/D
APPIAN, CLAY	B	ARMENIA	D	ATHELWOLD	B	BAAHISH	B	BARBAROSA	D
SUBSTRATUM		ARMESA	B	ATHENA	B	BABB	B	BARBARY	D
APPIAN, RECLAIMED	C	ARMIESBURG	B	ATHERTON	B/D	BABBINGTON	B	BARBERT	D
APPLEBUSH	B	ARMIJO	D	ATHOL	B	BABELTHUAP	B	BARBOUR	B
APPLETON	C	ARMINGTON	D	ATKINS	D	BACA	C	BARBOURVILLE	B
APPLING	B	ARMISTEAD	C	ATKINSON	B	BACH	B/D	BARCAVE	B
APRON	B	ARMITAGE	C	ATLAS	D	BACHUS	C	BARCLAY	C
APT	C	ARMO	B	ATLEE	C	BACKBAY	D	BARCO	B
APTAKISIC	B	ARMONA	D	ATLOW	D	BACKBONE	B	BARCUS	A
APTOS	C	ARMOUR	B	ATMORE	B/D	BACONA	B	BARD	D
AQUILLA	A	ARMSTER	C	ATOKA	C	BADAXE	B	BARDEN	C
AQUINAS	C	ARMSTRONG	C	ATONIC	B	BADENAUGH	B	BARDLEY	C
ARADA	B	ARMUCHEE	C	ATRAC	B	BADGE	B	BARELA	C
ARAGON	C	ARNEGARD	B	ATRING	B	BADGERTON	B	BARFIELD	D
ARAMBURU	C	ARNHEIM	D	ATRYPA	D	BADIN	C	BARFUSS	B
ARANSAS	D	ARNO	D	ATSION	B/D	BADITO	C	BARGE	C
ARAPAHOE	B/D	ARNOLD	B	ATSION, TIDE	D	BADO	D	BARIO	B
ARAPIEN	C	ARNOT	C/D	FLOODED		BADUS	C/D	BARISHMAN	C
ARATA	C	AROL	D	ATTER	A	BAGARD	B	BARKCAMP	B
ARAVAIPA	C	AROSA	C	ATTERBERRY	B	BAGDAD	B	BARKERVILLE	C
ARAVE	D	ARP	C	ATTEWAN	A	BAGGOTT	D	BARKLEY	C
ARAVETON	B	ARRADA	D	ATTEWAN,	B	BAGLEY	B	BARKOF	D
ARBELA	C	ARRASTRE	B	MODERATELY SLOW		BAHEM	B	BARLING	C
ARBIDGE	C	ARREDONDO	A	PERM		BAILE	D	BARNABE	D
ARBOLES	C	ARRIBA	C	ATTEWAN, WET	D	BAILEGAP	B	BARNARD	C
ARBONE	B	ARRINGTON	B	ATTICA	B	BAILING	C	BARNES	B
ARBOR	B	ARRIOLA	D	ATTOYAC	B	BAINVILLE	C	BARNESTON	A
ARBUCKLE	B	ARROLIME	C	ATWATER	B	BAIRD HOLLOW	C	BARNESTON, STONY	A
ARBUCKLE, CLAYEY	B	ARRON	D	ATWELL	D	BAJURA	D	BARNESTON,	B
SUBSTRATUM		ARROWHEAD	C	ATWOOD	B	BAKEOVEN	D	NONGRAVELLY	
ARBUCKLE, WET	C	ARROYADA	D	AU GRES	B	BAKER	C	BARNEY	D
ARBUCKLE, GRAVELLY	B	ARROYO SECO	B	AUBARQUE	D	BAKERSVILLE	D	BARNHARDT	B
ARBURUA	B	ARSITE	D	AUBBEENAUBBEE	B	BALAAN	B	BARNMOT	D
ARBUS	B	ARTA	C	AUBERRY	B	BALCOM	B	BARNSDALL	B
ARCETTE	B	ARTESIA	D	AUBREY	C	BALD	C	BARNUM	B
ARCH	B	ARTESIAN	D	AUBURN	D	BALDER	D	BARODA	D
ARCHABAL	B	ARTHOC	B	AUBURNDALE	B/D	BALDHILL	B	BAROID	A
ARCHER	C	ARTOIS	C	AUFCO	D	BALDMOUNTAIN	B	BAROID, WET	D
ARCHERDALE	C	ARUJO	B	AUGGIE	B	BALDOCK	D	BARRADA	D
ARCHES	D	ARUNDEL	C	AUGSBURG	B/D	BALDOCK, GRAVELLY	C	BARRE	D
ARCHIN	D	ARVADA	D	AUGUSTA	C	SUBSTRATUM,		BARRETT	D
ARCHULETA	D	ARVANA	C	AUGUSTINE	B	DRAINED		BARRIER	D
ARCIA	C	ARVESON	B/D	AULD	D	BALDOCK, DRAINED	C	BARRINGTON	B
ARCO	C	ARVILLA	A	AURA	B	BALDWIN	D	BARRON	B
ARCO, DRAINED	B	ARVIN	B	AURELIUS	B/D	BALDY	B	BARRONETT	B/D
ARD	C	ARZO	D	AURORA	C	BALE	C	BARRY	B/D
ARDENMONT	B	ASA	B	AUSTIN	C	BALE, WET	D	BARSAC	C
ARDENVOIR	B	ASCALON	B	AUSTWELL	D	BALLAHACK	D	BARSHAAD	D
ARDILLA	C	ASCAR	C	AUT	C	BALLARD	B	BART	B
ARDIVEY	B	ASCHOFF	B	AUTOMBA	B	BALLER	D	BARTINE	C
ARDNAS	B	ASH SPRINGS	B/C	AUTRYVILLE	A	BALLINGER	D	BARTLE	D
ARDTOO	B	ASHBON	D	AUXVASSE	D	BALLTOWN	D	BARTLEY	C
ARECIBO	A	ASHCROFT	B	AUZQUI	B	BALLY	C	BARTO	D
AREDALE	B	ASHDALE	B	AVA	B	BALM	D	BARTOME	D
ARENA	D	ASHDOWN	B	AVA	C	BALMAN	C	BARTON	B
ARENA, DRAINED	C	ASHE	B	AVALON	B	BALMAN, DRAINED	B	BARTONFLAT	B
ARENALES	A	ASHER	C	AVAR	D	BALMORHEA	C	BARVON	B
ARENDTSVILLE	B	ASHFORD	D	AVAWATZ	A	BALON	B	BASCAL	B
ARENOSA	A	ASHFORK	D	AVENAL	B	BALTIC	D	BASCO	C
ARENZVILLE	B	ASHGROVE	D	AVILLA	B	BALTIMORE	B	BASCOM	B
ARGENT	D	ASHIPPUN	C	AVIS	A	BAMA	B	BASCOVY	D
ARGENTA	C	ASHKUM	B/D	AVOCA	B	BAMBER	B	BASEHOR	D
ARGONAUT	D	ASHLAR	B	AVON	C	BAMOS	C	BASHAW	D
ARGYLE	B	ASHLEY	B	AVONBURG	D	BANTUSH	B	BASHER	B
ARIEL	C	ASHLO	B	AVONDA	B	BANBURY	D	BASILE	D
ARIKARA	B	ASHTON	B	AVONDALE	B	BANCAS	C	BASIN	C
ARIMO	B	ASHUE	B	AVONVILLE	B	BANCROFT	B	BASINGER	B/D
ARIPEKA	C	ASHUELOT	D	AVTABLE	D	BANCY	D	BASKET	B
ARIS	D	ASHWOOD	C	AWBRIG	D	BANDAG	B	BASSEL	B
ARISPE	C	ASKEW	C	AXIS	D	BANDERA	B	BASSETT	B
ARIZO	A	ASOTIN	C	AXTELL	D	BANDID	B	BASSFIELD	B
ARKABUTLA	C	ASPARAS	B	AYAR	D	BANDON	C	BASTIAN	C
ARKANA	C	ASPEN	B	AYCOCK	B	BANE	A	BASTON	C
ARKAQUA	C	ASPERMONT	B	AYDELOTTE	D	BANGO	B	BASTROP	B
ARKONA	B	ASPERSON	C	AYERSVILLE	B	BANGOR	B	BASTSIL	B
ARKPORT	B	ASSININS	B	AYLMER	A	BANGSTON	A	BATA	B
ARKSON	B	ASSINNIBOINE	B	AYNOR	B/D	BANIDA	D	BATAN	B
ARKTON	C	ASSUMPTION	B	AYON	B	BANKARD	A	BATAVIA	B
ARLAND	B	ASTA	B	AYR	B	BANKS	A	BATENAN	C
ARLE	C	ASTATULA	A	AYRES	D	BANLIC	C	BATES	B
ARLINGTON	C	ASTOR	D	AYRSHIRE	C	BANNEL	B	BATESVILLE	C
ARLINGTON, THICK	B	ASTORIA	B	AYSEES	B	BANNER	C	BATH	C
SOLUM		ATASCO	C	AZAAR	C	BANNING	C	BATTERSON	D
ARLO	B	ATASCOSA	D	AZELTINE	B	BANNOCK	B	BATTLE CREEK	C
ARLOVAL	A	ATCHEE	D	AZTALAN	C	BAPOS	D	BATZA	D

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TABLE 7.1--HYDROLOGIC GROUPS OF THE SOILS OF THE UNITED STATES

BAUDETTE	B	BEMANIN	B	BERNAL	D	BINGHAMPTON	B	BLAND	C
BAUER	C	BEHEMOTOSH	C	BERNALDO	B	BINGHAMVILLE	D	BLANDING	B
BAUMGARD	B	BEHRING	D	BERNARD	D	BINNA	B	BLANEY	B
BAUSCHER	B	BEISIGL	A	BERNARDINO	C	BINNSVILLE	D	BLANKET	C
BAXENDALE	B	BEJE	D	BERNARDSTON	C	BINS	B	BLANTON	A
BAXTER	B	BEJUCOS	B	BERNHILL	B	BINTON	C	BLANYON	C
BAXTERVILLE	B	BELAIN	C	BERNICE	A	BIOYA	B	BLASDELL	A
BAYAMON	B	BELATE	B	BERNING	C	BIPPUS	B	BLASE	C
BAYARD	B	BELCHER	D	BERNOW	B	BIRCHBAY	B	BLASINGAME	C
BAYBORO	D	BELDEN	C	BERRYLAND	B/D	BIRCHWOOD	C	BLAYDEN	D
BAYERTON	C	BELDING	B	BERRYMAN	C	BIRDOW	B	BLAZON	D
BAYFIELD	C	BELEN	D	BERSON	B	BIRDS	C/D	BLEAKWOOD	C
BAYFIELD, WET	D	BELFAST	B	BERTAG	C	BIRDSALL	D	BLEDSOE	C
BAYLIS	B	BELFIELD	C	BERTELSON	B	BIRDSBORO	B	BLEIBLERVILLE	D
BAYMEADE	A	BELFORE	B	BERTHOUD	B	BIRDSLEY	D	BLENCOE	D
BAYOU	D	BELGRADE	B	BERTIE	B	BIRDSVIEW	A	BLEND	D
BAYSHORE	D	BELHAVEN	D	BERTOLOTTI	B	BIRKBECK	B	BLENDON	B
BAYSHORE,	B	BELINDA	D	BERTRAM	B	BIRMINGHAM	B	BLETHEN	B
MODERATELY WET		BELJICA	B	BERTRAND	B	BIRNEY	B	BLEVINS	B
BAYTOWN	B	BELK	D	BERVILLE	B/D	BIROME	C	BLEVINTON	B
BAYUCOS	D	BELKNAP	C	BERWOLF	B	BISBEE	A	BLEWETT	D
BAYVI	D	BELLAVISTA	C	BERZATIC	D	BISCAY	B/D	BLICHTON	D
BAYVIEW	D	BELLE	B	BESEMAN	A/D	BISGANI	C	BLINN	C
BAYWOOD	A	BELLECHESTER	A	BESNER	B	BISHOP	D	BLOMFORD	B/D
BAZETTE	C	BELLEVILLE	B/D	BESSEMER	C	BISODDI	D	BLOOM	D
BAZILE	B	BELLEVILLE, PONDED	D	BESSIE	D	BISPING	B	BLOOMFIELD	A
BEACH	D	BELLEVUE	B	BESTROM	C	BISSELL	B	BLOOMING	B
BEAD	C	BELLICUM	B	BETHANY	C	BISSONNET	D	BLOOMSDALE	B
BEADLE	C	BELLINGHAM	D	BETHERA	D	BIT	C	BLOOR	C
BEALES	B	BELLINGHAM, PONDED	D	BETHESDA	C	BITTER	B	BLOOR, NONFLOODED	C
BEAMTON	C	BELLINGHAM,	C	BETIS	A	BITTER SPRING	B	BLOOR, GRAVELLY	D
BEANFLAT	C	DRAINED		BETTERAVIA	C	BITTERROOT	C	SUBSTRATUM	
BEANO	D	BELLPASS	D	BETTS	B	BITTERWATER	B	BLOUNT	C
BEAR BASIN	B	BELLPINE	C	BEULAH	B	BITTON	B	BLOWERS	B
BEAR CREEK	B	BELMEAR	D	BEVENT	A	BIVANS	D	BLUCHER	C
BEAR LAKE	D	BELMILL	B	BEVERIDGE	D	BIXBY	B	BLUE EARTH	B/D
BEAR PRAIRIE	B	BELMONT	B	BEVERLY	A	BIXLER	C	BLUE EARTH	B/D
BEARDALL	C	BELMORE	B	BEW	C	BJORK	C	BLUE EARTH,	D
BEARDEN	C	BELPRE	C	BEWLEYVILLE	B	BLACHLY	C	SLOPING	
BEARDSLEY	C	BELTED	D	BEXAR	D	BLACK BUTTE	B	BLUE LAKE	A
BEARDSTOWN	C	BELTON	C	BEZZANT	B	BLACK CANYON	D	BLUE STAR	B
BEARMOUTH	A	BELTRAMI	B	BIBB	C	BLACK CANYON,	C	BLUEBELL	B
BEARPAW	C	BELTSVILLE	C	BICE	B	DRAINED		BLUEBELL, COOL	C
BEARSKIN	D	BELUGA	D	BICKERDYKE	D	BLACK RIDGE	D	BLUECHIEF	B
BEARTRAP	B	BELVOIR	C	BICKLETON	B	BLACKA	C	BLUECREEK	C
BEARVILLE	C	BELZAR	C	BICKMORE	C	BLACKBURN	B	BLUEDOME	C
BEARWALLOW	B	BEN LOMOND	B	BICONDOA	D	BLACKETT	B	BLUEFLAT	C
BEASLEY	C	BENCLARE	C	BICONDOA, LOAMY	D	BLACKFOOT	C	BLUEGROVE	C
BEASON	C	BENCO	B	SUBSTRATUM		BLACKFOOT,	B	BLUEHILL	C
BEATRICE	D	BENEVOLA	C	BICONDOA, DRAINED	C	FREQUENTLY		BLUEHON	B
BEAUCOUP	B/D	BENEWAH	D	BIDDEFORD	D	FLOODED		BLUEHON, WARM	C
BEAUFORD	D	BENFIELD	C	BIDDLEMAN	B	BLACKFOOT, DRAINED	B	BLUEJOINT	B
BEAUMONT	D	BENGAL	C	BIDMAN	C	BLACKHALL	D	BLUENOSE	B
BEAUREGARD	C	BENGE	B	BIDWELL	C	BLACKHAMMER	B	BLUEPOINT	A
BEAUSITE	B	BENHAM	B	BIEBER	D	BLACKHAWK	D	BLUERIM	C
BEAUVAIS	B	BENIN	D	BIEDELL	D	BLACKHOOF	D	BLUESLIDE	D
BEAVERCREEK	B	BENITO	D	BIENVILLE	A	BLACKLEED	B	BLUESPRIN	C
BEAVERDAM	C	BENJAMIN	D	BIG BLUE	D	BLACKLEG	C	BLUESTONE	D
BEAVERELL	B	BENKLIN	C	BIG HORN	C	BLACKLOCK	D	BLUEWING	A
BEAVERLAND	B	BENMAN	C	BIG TIMBER	D	BLACKMAN	C	BLUFF	D
BEAVERTON	B	BENNDALE	B	BIGBEE	A	BLACKMOUNT	B	BLUFFDALE	C
BECKER	B	BENNINGTON	C	BIGBROWN	C	BLACKNOLL	C	BLUFFTON	C/D
BECKET	C	BENRIDGE	B	BIGELOW	B	BLACKOAR	B/D	BLUFORD	C
BECKLEY	A	BENSLEY	B	BIGETTY	B	BLACKPIPE	C	BLUM	C
BECKLEY, STONY	B	BENSON	C/D	BIGFORK	C	BLACKROCK	B	BLY	B
BECKMAN	D	BENTEEN	B	BIGNELL	B	BLACKSPAR	D	BLYBURG	B
BECKS	C	BENZ	D	BIGRIVER	C	BLACKSTON	B	BLYTHE	D
BECKTON	D	BEOR	D	BIGSPRINGS	C	BLACKTOP	D	BOARDMAN	D
BECKWITH	D	BEOSKA	B	BIGWIN	C	BLACKWATER	D	BOARDTREE	C
BECKWOURTH	C	BEOTIA	B	BIJOU	B	BLACKWELL	D	BOASH	D
BECREEK	B	BEOWAWE	C	BILBO	C	BLADEN	D	BOAZ	C
BEDELL	B	BEQUINN	B	BILGER	D	BLAG	D	BOBBITT	C
BEDEN	D	BERCUMB	B	BILLETT	B	BLAGO	D	BOBILLO	A
BEDFORD	C	BERDA	B	BILLINGS	C	BLAINE	C	BOBS	D
BEDINGTON	B	BEREA	C	BILLINGS,	B	BLAIR	C	BOBTAIL	C
BEDKE	B	BERENICETON	B	MODERATELY SLOW		BLAIRTON	C	BOBTOWN	B
BEDNER	C	BERGLAND	D	PERM		BLAKABIN	C	BOCA	B/D
BEDSTEAD	C	BERGQUIST	B	BILLINGS,	C	BLAKE	B	BOCA, DEPRESSIONAL	B/D
BEDWYR	D	BERGSTROM	B	SALINE-ALKALI		BLAKELAND	A	BOCA, TIDAL	D
BEE	B	BERGSVIK	D	BILLYCREEK	C	BLAKENEY	C	BOCA, SLOUGH	B/D
BEEBE	A	BERINO	B	BILLYHAW	D	BLALOCK	D	BOCK	B
BEECHER	C	BERIT	D	BILTMORE	A	BLAMER	C	BOCKER	D
BEEK	C	BERKS	C	BIMMER	D	BLANCA	B	BOCKSTON	B
BEEKMAN	C	BERKSHIRE	B	BINCO	D	BLANCHARD	A	BODE	B
BEELINE	D	BERLAKE	B	BINDLE	B	BLANCHE	B	BODELL	D
BEENOM	D	BERLIN	C	BINFORD	B	BLANCHESTER	B/D	BODEN	C
BEETVILLE	B	BERMESA	C	BINGER	B	BLANCHO	C	BODENBURG	B
BEGAY	B	BERMUDIAN	B	BINGHAM	B	BLANCOT	B	BODINE	B

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BODO	C	BORDEAUX	B	BRAHAN	B	BRISKY	D	BUCAN	C
BODORUMPE	C	BORDEN	B	BRAILSFORD	C	BRISTOW	D	BUCAN, STONY	C
BOEL	A	BORDER	B	BRAINERD	C	BRITTO	D	BUCAN, GRAVELLY	D
BOEL, OVERWASH	C	BORGES	D	BRALLIER	D	BRITWATER	B	BUCHANAN	C
BOELUS	A	BORIANA	D	BRAN	C	BROAD	C	BUCHEL	D
BOERNE	B	BORKY	C	BRAMARD	B	BROAD CANYON	B	BUCHENAU	C
BOESEL	B	BORNSTEDT	C	BRAMLETT	D	BROADALBIN	C	BUCHENAU, SALINE	C
BOETTCHER	C	BORO	D	BRAMWELL	C	BROADAX	B	BUCHENAU, THICK	B
BOGAN	C	BOROBEY	C	BRANCH	B	BROADBROOK	C	SOLUM	
BOGART	B	BORREGO	D	BRAND	D	BROADHEAD	C	BUCKCREEK	C
BOGGS	C	BORTH	C	BRANDENBURG	A	BROADHURST	D	BUCKEYE	B
BOGGY	C	BORUP	B/D	BRANDON	B	BROADMOOR	C	BUCKHALL	B
BOGRAP	B	BORVANT	D	BRANDYWINE	C	BROADUS	B	BUCKHOUSE	B
BOGUE	D	BOSANKO	D	BRANFORD	B	BROADWELL	B	BUCKLAND	C
BOGUS	C	BOSCO	B	BRANHAM	C	BROCK	D	BUCKLE	B
BOHANNON	C	BOSKET	B	BRANTFORD	B	BROCKET	C	BUCKLEBAR	B
BOHEMIAN	B	BOSLER	B	BRANTLEY	C	BROCKLISS	B	BUCKLEY	D
BOHICKET	D	BOSQUE	B	BRANYON	D	BROCKMAN	C	BUCKLEY, DRAINED	C
BOHNA	B	BOSSBURG	D	BRASHEAR	C	BROCKO	B	BUCKLICK	C
BOHNLY	D	BOSSBURG, DRAINED	C	BRASSFIELD	B	BROCKPORT	D	BUCKLON	D
BOHNSACK	B	BOSTON	C	BRATTON	B	BROCKROAD	C	BUCKNELL	D
BOISTFORT	B	BOSTRUM	C	BRAUN	B	BROCKSBURG	B	BUCKNEY	B
BOJAC	B	BOSTWICK	B	BRAVANE	D	BROCKTON	D	BUCKPEAK	B
BOJO	D	BOSVILLE	D	BRAXTON	C	BROCKWAY	B	BUCKS	B
BOLAN	B	BOSWELL	D	BRAY	D	BROCKWELL	B	BUCKSKIN	C
BOLAR	C	BOSWORTH	D	BRAYTON	C	BRODALE	C	BUCKTON	B
BOLO	B	BOTELLA	B	BRAZITO	A	BRODY	C	BUDE	C
BOLES	C	BOTHWELL	B	BRAZON	C	BROGAN	B	BUDIHOL	D
BOLFAR	C	BOTTINEAU	C	BRAZORIA	D	BROGDON	B	BUELL	B
BOLIO	D	BOTTLE	C	BRECKENRIDGE	B/D	BROLLIAR	D	BUENA VISTA	B
BOLIVAR	B	BOTTLEROCK	C	BRECKNOCK	B	BRONER	C	BUFFARAN	D
BOLLING	C	BOUFLAT	C	BRECKSVILLE	C	BRONIDE	B	BUFFINGTON	B
BOLSA	C	BOULDER	B	BREECE	B	BRONO	B	BUFFMEYER	B
BOLTON	B	BOULDER LAKE	D	BREGAR	D	BRONAUGH	B	BUFFORK	C
BOLTUS	D	BOULDER POINT	B	BREMER	C	BRONCHO	B	BUFTON	C
BONAR	C	BOULDIN	B	BREMER, SANDY	B	BRONSON	B	BUHRIG	C
BONBADIL	D	BOULFLAT	C	SUBSTRATUM		BRONTE	C	BUICK	C
BOMBAY	B	BOUNDARY	B	BRENO	C	BROOKE	D	BUIST	B
BON	B	BOURBON	B	BRENS	A	BROOKFIELD	B	BUKO	B
BONAIR	D	BOURNE	C	BRENDA	C	BROOKINGS	B	BUKO, WET	C
BONANZA	B	BOUSIC	D	BRENHAM	C	BROOKLYN	C/D	BUKREEK	B
BONAPARTE	A	BOW	C	BRENNAN	B	BROOKMAN	D	BULAKE	D
BOND	D	BOWBAC	C	BRENNER	D	BROOKSHIRE	C	BULKLEY	C
BONOFARM	D	BOWBELLS	B	BRENT	D	BROOKSIDE	C	BULL RUN	B
BONONAN	D	BOWDISH	C	BRENTON	B	BROOKSTON	B/D	BULL RUN, HARDPAN	C
BONDRANCH	D	BOWDISH, DRY	B	BRENTWOOD	B	BROOKSTON,	B/D	SUBSTRATUM	
BONDUEL	C	BOWDLE	B	BRESSA	C	OVERWASH		BULL TRAIL	B
BONE	D	BOWDOIN	D	BRESSER	B	BROOKSTON, STONY	D	BULLARDS	B
BONEEK	B	BOWORE	C	BREVARD	B	BROOKSVILLE	D	BULLION	D
BONFIELD	B	BOWEN	B	BREVORT	B/D	BROOME	B	BULLNEL	C
BONFRI	C	BOWERS	C	BREW	C	BROPHY	A/D	BULLOCK	D
BONG	A	BOWES	B	BREWER	C	BROSE	D	BULLREY	B
BONHAN	C	BOWIE	B	BREWSTER	D	BROSELEY	B	BULLUMP	B
BONIFAY	A	BOWMAN	C	BREWTON	C	BROSS	C	BULLWINKLE	D
BONILLA	B	BOWMANSVILLE	B/D	BRIBUTTE	D	BROUGHTON	D	BULLY	B
BONITA	D	BOWNS	C	BRICKEL	C	BROWARD	C	BULOW	A
BONN	D	BOWSTRING	A/D	BRICKTON	C	BROWER	B	BUNCOMBE	A
BONNEAU	A	BOXFORD	C	BRICO	C	BROWNDELL	D	BUNDO	B
BONNELL	C	BOXVILLE	C	BRIDGE	C	BROWNELL	B	BUNDORF	D
BONNER	B	BOXWELL	C	BRIDGECREEK	C	BROWNFIELD	A	BUNDY	C
BONNET	B	BOY	B	BRIDGEHAMPTON	B	BROWNLEE	B	BUNDYMAN	C
BONNEVILLE	A	BOYCE	B/D	BRIDGEPORT	B	BROWNRIGG	D	BUNEJUG	C
BONNIE	C/D	BOYD	D	BRIDGER	B	BROWNSCOMBE	C	BUNGAY	C/D
BONNYDOON	D	BOYER	B	BRIDGESON	D	BROWNSTO	B	BUNKER	B
BONO	D	BOYKIN	B	BRIDGESON, DRAINED	C	BROWNSVILLE	C	BUNKERHILL	D
BONSALL	D	BOYLE	D	BRIDGET	B	BROWNTON	C/D	BUNKY	C
BONTA	B	BOYSAG	D	BRIEDWELL	B	BROXON	B	BUNNELL	B
BONTI	C	BOYSEN	D	BRIEF	B	BROYLES	B	BUNTINGVILLE	C
BONWIER	C	BOZE	B	BRIGGS	A	BRUBECK	D	BUNYAN	B
BONWIER, GRADED	D	BOZEMAN	B	BRIGGSDALE	C	BRUCE	B/D	BURBANK	A
BOOFORD	C	BRACE	B	BRIGGSVILLE	C	BRUFFY	B	BURCH	B
BOOKCLIFF	B	BRACEVILLE	C	BRIGHTON	B/D	BRUIN	B	BURCHAN	B
BOOKER	D	BRACKETT	C	BRIGHTWOOD	B	BRUNCAN	D	BURCHARD	B
BOOKOUT	C	BRAD	D	BRILEY	B	BRUNDAGE	D	BURDETT	C
BOOKWOOD	B	BRADDOCK	B	BRILL	B	BRUNEEL	D	BUREN	C
BOOMER	B	BRADEN	B	BRILLIANT	B	BRUNO	A	BURGESS	C
BOONTOWN	D	BRADENTON	D	BRIMFIELD	C/D	BRUNSWICK	B	BURGI	B
BOONE	A	BRADENTON,	B/D	BRIMLEY	B	BRUNZELL	B	BURKE	C
BOONESBORO	B	LIMESTONE		BRIMSTONE	D	BRUSSETT	B	BURKETOWN	C
BOONTON	C	SUBSTRATUM		BRINEGAR	B	BRYAN	A	BURKEVILLE	D
BOOTH	C	BRADENTON, FLOODED	D	BRINKERT	C	BRYANT	B	BURKHARDT	B
BOOTHBAY	C	BRADER	D	BRINKERTON	D	BRYARLY	D	BURLEIGH	A/D
BOOTJACK	D	BRADSHAW	B	BRINNUN	D	BRYCAN	B	BURLESON	D
BOOTS	A/D	BRADSON	B	BRINNUN, DRAINED	C	BRYCE	D	BURLEWASH	D
BOQUILLAS	C	BRADWAY	D	BRIONES	B	BRYNAN	B	BURLINGTON	A
BORACHO	C	BRADY	B	BRIOS	A	BRYSTAL	B	BURNAH	D
BORAH	C	BRADYVILLE	C	BRISCOT	D	BUB	C	BURNAC	D
BORDA	D	BRAGG	C	BRISCOT, DRAINED	C	BUBUS	B	BURNBOROUGH	B

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TABLE 7.1--HYDROLOGIC GROUPS OF THE SOILS OF THE UNITED STATES

BURNEL	C	CALCROSS	C	CANISTEO, SANDY	B/D	CARSITAS, COBBLY	A	CATTCREEK	B
BURNETTE	C	CALO	C	SUBSTRATUM		CARSITAS,	A	CATTO	D
BURNHAM	D	CALDER	D	CANLON	D	NONGRAVELLY		CAUDLE	C
BURNSIDE	B	CALDWELL	C	CANNELL	B	CARSON	D	CAUSEWA	C
BURNSVILLE	B	CALDWELL, DRAINED	B	CANNING	B	CARSTAIRS	A	CAUSEY	B
BURNT LAKE	A	CALE	B	CANNON	B	CARSTUMP	C	CAVAL	B
BURR	D	CALEAST	C	CANOE	B	CART	B	CAVANAUGH	C
BURRITA	D	CALEB	B	CANOVA	B/D	CARTAGENA	D	CAVE	D
BURROWSVILLE	C	CALEDONIA	B	CANTALA	B	CARTECAY	C	CAVELT	D
BURSON	C	CALENDAR	C	CANTEY	D	CARTER	D	CAVENDISH	B
BURT	D	CALHI	A	CANTON	B	CARTERET	D	CAVO	D
BURTON	B	CALHOUN	D	CANTON BEND	C	CARTHAGE	B	CAVODE	C
BURWELL	C	CALICO	C	CANTRIL	B	CARUSO	C	CAVOUR	D
BUSBY	B	CALICOTT	A	CANTUA	B	CARUTHERSVILLE	B	CAYA	D
BUSE	B	CALIFON	C	CANTUCHE	D	CARVER	A	CAYAGUA	C
BUSHER	B	CALIMUS	B	CANUTIO	B	CARWILE	D	CAYUGA	C
BUSHMAN	B	CALITA	B	CANYON	D	CARYTOWN	D	CAYUSE	B
BUSHNELL	C	CALIZA	B	CAPAC	C	CARYVILLE	B	CAZADERO	C
BUSHVALLEY	D	CALKINS	C	CAPAY	D	CASA GRANDE	C	CAZENOVIA	B
BUSKA	B	CALLABO	C	CAPE	D	CASABONNE	B	CEBOLIA	C
BUSSY	C	CALLAHAN	D	CAPE FEAR	D	CASAGA	C	CEBONE	C
BUSTER	B	CALLAN	C	CAPEHORN	D	CASCADE	C	CEBOYA	C
BUTCHE	D	CALLEGUAS	D	CAPERS	D	CASCAJO	A	CECIL	B
BUTLER	D	CALLINGS	D	CAPERTON	D	CASCILLA	B	CEDA	B
BUTLERTOWN	C	CALLISBURG	C	CAPHOR	C	CASCO	B	CEDAR BUTTE	D
BUTTERFIELD	C	CALLOWAY	C	CAPILLO	C	CASE	B	CEDAR MOUNTAIN	D
BUTTERS	B	CALMAR	B	CAPISTRANO	B	CASEY	D	CEDARAN	D
BUTTON	D	CALODO	C	CAPITAN	D	CASHEL	C	CEDARBLUFF	C
BUTTONWILLOW	D	CALOUSE	B	CAPJAC	C	CASHION	D	CEDARGAP	B
BUXIN	D	CALPAC	B	CAPLES	D	CASHMERE	B	CEDARHILL	B
BUXTON	C	CALPINE	B	CAPLES, DRAINED	C	CASHMONT	B	CEDARPASS	B
BYARS	D	CALROY	B	CAPONA	C	CASITO	D	CEDONIA	B
BYBEE	D	CALVERTON	C	CAPPS	B	CASMOS	D	CELACY	C
BYLER	C	CALVIN	C	CAPSHAW	C	CASPAR	B	CELESTE	D
BYLO	B	CALVISTA	D	CAPTINA	C	CASPIANA	B	CELETON	D
BYNUM	C	CALWOODS	D	CAPTIVA	B/D	CASS	B	CELINA	C
BYRAM	C	CAMAGUEY	D	CAPULIN	B	CASSIA	C	CELIO	D
BYRNIE	D	CAMARGO	B	CARACOLES	D	CASSIA, MODERATELY	B	CELLAR	D
CABALLO	B	CAMARILLO	C	CARADAN	D	WELL DRAINED		CELSOSPRINGS	C
CABARTON	D	CAMARILLO, DRAINED	B	CARALAMPI	B	CASSIRO	C	CEMBER	C
CABBA	D	CAMARILLO, FLOODED	C	CARBENGLE	B	CASSOLARY	C	CENCOVE	B
CABBART	D	CAMAS	A	CARBO	C	CASTAIC	C	CENIZA	B
CABEZON	D	CAMATTA	D	CARBOL	D	CASTALIA	C	CENTENARY	B
CABIN	B	CAMBARGE	B	CARBONDALE	A/D	CASTANA	B	CENTER	C
CABINET	C	CAMBERN	C	CARCITY	D	CASTELL	C	CENTER CREEK	C
CABLE	B/D	CAMBERT	C	CARDIFF	B	CASTELLEIA	B	CENTERFIELD	B
CABO ROJO	C	CAMBETH	C	CARDIGAN	B	CASTELLO	B	CENTERVILLE	D
CABOOSE	B	CAMBRIDGE	C	CARDINGTON	C	CASTEPHEN	C	CENTISSIMA	B
CABOT	D	CAMDEN	B	CARDON	D	CASTILE	B	CENTRAL POINT	B
CABSTON	C	CAMELBACK	B	CAREFREE	D	CASTINO	C	CENTRALIA	B
CACHE	D	CAMERON	D	CAREY	B	CASTINO, NONSTONY	D	CERBAT	D
CACIQUE	C	CAMILLUS	B	CAREY LAKE	B	CASTLE	D	CERESCO	B
CACTUSFLAT	C	CAMINO	C	CARGILL	C	CASTLEVALE	D	CERLIN	C
CADDO	D	CAMPBELL, MUCK	C	CARIBEL	B	CASTNER	D	CERRILLOS	B
CADEVILLE	D	SUBSTRATUM		CARIBOU	B	CASTO	C	CERRO	C
CADILLAC	A	CAMPBELL, DRAINED	B	CARIOCA	B	CASTON	B	CESTNIK	C
CADIZ	B	CAMPBELLTON	C	CARJO	C	CASTRO	D	CETRACK	B
CADMUS	B	CAMPIA	B	CARLIN	D	CASTROVILLE	B	CHACON	D
CADOMA	C	CAMPO	C	CARLINTON	C	CASUSE	D	CHAD	C
CAGEY	D	CAMPONE	C	CARLISLE	A/D	CASWELL	B	CHAFFEE	C
CAGEY, DRAINED	C	CAMPSPASS	B	CARLITO	D	CATALINA	B	CHAGRIN	B
CAGLE	C	CAMPUS	B	CARLOS	A/D	CATALPA	C	CHAIRES	B/D
CAGUABO	D	CAMRODEN	C	CARLOW	D	CATAMOUNT	C	CHAIX	B
CAGWIN	B	CANA	C	CARLSBAD	C	CATANO	A	CHALCO	D
CAHABA	B	CANAAN	C	CARLSBORG	A	CATARACT	B	CHALFONT	C
CAHONA	B	CANADIAN	B	CARLSON	B	CATARINA	D	CHALMERS	B/D
CAIO	B	CANADICE	D	CARLSTROM	C	CATASKA	D	CHAMA	B
CAINHOY	A	CANALOU	B	CARLTON	B	CATAULA	B	CHAMATE	B
CAIRO	D	CANANDAIGUA	D	CARMACK	B	CATCHELL	C	CHAMBERINO	C
CAJALCO	C	CANASERAGA	C	CARMEL	C	CATELLI	B	CHAMISE	D
CAJETE	A	CANAVERAL	C	CARMI	B	CATER	B	CHAMOKANE	B
CAJON, OVERWASH	A	CANBURN	D	CARMODY	C	CATH	C	CHAMPAGNE	B
CAJON, LOAMY	A	CANDELERO	C	CARNAGE	D	CATHARPIN	C	CHAMPION	B
SUBSTRATUM		CANDERLY	B	CARNASAW	C	CATHAY	C	CHANAC	B
CAJON, SILTY	B	CANDLER	A	CARNEGIE	C	CATHCART	B	CHANCE	D
SUBSTRATUM		CANDOR	A	CARNERO	D	CATHEDRAL	D	CHANCELLOR	C
CAJON,	B	CANE	C	CARNEY	D	CATHERINE	B/D	CHANDLER	B
SALINE-ALKALI		CANEADEA	D	CAROLINE	C	CATHLAMET	B	CHANEY	C
CAJON, GRAVELLY	A	CANEEK	B	CAROLLO	D	CATHRO	A/D	CHANNAHON	D
CAJON, COOL	A	CANELO	D	CARON	A/D	CATILLA	B	CHANNING	B
CAJON, WARM	A	CANEST	D	CARPENTER	B	CATLA	D	CHANTA	B
CALABASAS	B	CANEYVILLE	C	CARR	B	CATLETT	C/D	CHANTIER	D
CALAMINE	D	CANEZ	B	CARRACAS	D	CATLIN	B	CHAPERTON	C
CALAPOOYA	C	CANFIELD	C	CARRIZALES	A	CATMAN	D	CHAPIN	C
CALAVERAS	B	CANISTEO	B/D	CARRIZO	A	CATNIP	D	CHAPMAN	B
CALAWAH	B	CANISTEO, PONDED	B/D	CARRYBACK	C	CATOCTIN	C	CHAPOT	B
CALCO	B/D	CANISTEO, STONY	D	CARSITAS	A	CATOOSA	B	CHAPPELL	A
CALCOUSTA	B/D			CARSITAS, WET	B	CATPOINT	A	CHARCOL	B

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CHARD	B	CHILCOTT	C	CINEBAR	B	CLOUDLAND	C	COLMOR	B
CHARDOTON	C	CHILGREN	C	CINNAMON	B	CLOUGH	D	COLO	B/D
CHARETTE	C	CHILHOWIE	C	CINTRONA	C	CLOVELLY	D	COLO, MAP<25	B/D
CHARGO	D	CHILI	B	CIPRIANO	D	CLOVER SPRINGS	B	COLO, NONFLOODED	B
CHARITON	C	CHILICOTAL	B	CIRCLEBACK	A	CLOVERDALE	D	COLOCKUM	B
CHARLEBOIS	B	CHILL	D	CIRCLEBAR	C	CLOVIS	B	COLOMA	A
CHARLEBOIS, WET	C	CHILLUM	B	CIRCLEVILLE	C	CLOWERS	B	COLOMBO	B
CHARLES	C	CHILMARK	C	CISCO	B	CLOWERS, WET	C	COLONA	C
CHARLESTON	C	CHILOQUIN	B	CISNE	D	CLOWFIN	B	COLONIE	A
CHARLEVOIX	B	CHILSON	D	CISPUS	A	CLUFF	C	COLONVILLE	C
CHARLOS	B	CHILTON	B	CITADEL	C	CLUNIE	D	COLORADO	B
CHARLOS, WET	B/D	CHIMAYO	D	CITICO	B	CLUROE	B	COLOROCK	D
CHARLOTTE	B/D	CHINE	C	CLACKAMAS	D	CLURO	B	COLOROW	B
CHARLTON	B	CHIMENEA		CLAIBORNE	B	CLYDE	B/D	COLOSO	D
CHASE	C	CHIMENEA, STONY	D	CLAIRE	A	CLYMER	B	COLOSSE	A
CHASEBURG	B	CHINCHALLO	D	CLAIREMONT	B	COACHELLA	B	COLP	C
CHASEVILLE	A	CHINIAK	A	CLALLAM	C	COACHELLA, WET	C	COLRAIN	B
CHASKA	B/D	CHINO	C	CLAM GULCH	D	COAHUILA	B	COLTER	B
CHASTAIN	D	CHINO, SALINE	C	CLAMO	C/D	COALBANK	B	COLTHORP	D
CHATBURN	B	CHINO,	C	CLANA	A	COALDRAW	D	COLTON	A
CHATCOLET	B	SALINE-ALKALI		CLANALPINE	C	COALMONT	C	COLTS NECK	B
CHATEAU	D	CHINO, DRAINED	B	CLANTON	C	COANO	C	COLUMBIA,	C
CHATFIELD	B	CHINOOK	B	CLAPPER	B	COARSEGOLD	C	MODERATELY WET	
CHATHAM	B	CHIPETA	D	CLAREMORE	D	COATSBURG	D	COLUMBIA, FLOODED	B
CHATSWORTH	D	CHIPLEY	C	CLARENCE	D	COBB	B	COLUMBIA, CLAY	C
CHATT	C	CHIPMAN	D	CLARENDON	C	COBBSFORK	D	SUBSTRATUM	
CHATUGE	D	CHIPOLA	A	CLARESON	C	COBEN	D	COLUMBIA,	B
CHAUMONT	D	CHIPPENY	D	CLAREVILLE	C	COBEY	B	PROTECTED	
CHAUNCEY	C	CHIPPEWA	D	CLARINDA	D	COBURG	C	COLUMBINE	A
CHAVIES	B	CHIREND	D	CLARION	B	COCHETOPA	C	COLUMBUS	C
CHAWANAKEE	C	CHIRICAHUA	D	CLARITA	D	COCHINA	D	COLUSA	C
CHAYSON	C	CHISCA	D	CLARK	B	COCHITI	C	COLVARD	B
CHAZOS	C	CHISNORE	D	CLARK FORK	A	COCOA	A	COLVILLE	D
CHEADLE	D	CHISOLM	A	CLARKRANGE	C	COCOLALLA	D	COLVILLE, DRAINED	C
CHEAHA	D	CHISPA	B	CLARKSBURG	C	COCOLALLA, DRAINED	C	COLVIN	C/D
CHECKETT	D	CHITINA	C	CLARKSDALE	C	CODORUS	C	COLVIN, SALINE	C
CHEDEHAP	B	CHITTUM	D	CLARKSVILLE	B	COE	A	COLVIN, PONDED	C/D
CHEDSEY	C	CHITWOOD	C	CLARNO	B	COEROCK	D	COLWOOD	B/D
CHEEBE	D	CHIVATO	D	CLATO	B	COFF	C	COLY	B
CHEEKTOWAGA	D	CHIWAWA	B	CLATSOP	D	COFFEEN	B	COLYER	D
CHEESEMAN	B	CHLORIDE	D	CLAVERACK	C	COGGON	B	COMAD	A
CHEHALEM	C	CHO	C	CLAVICON	C	COGNA	B	COMAR	B
CHEHALIS	B	CHOBEE	B/D	CLAWSON	C	COGSWELL	C	COMBE	B
CHEHULPUM	C	CHOCCOLOCCO	B	CLAYBURN	B	COHAGEN	D	COMBS	B
CHELAN	B	CHOCK	D	CLAYSPRINGS	D	COHASSET	B	COMER	B
CHELSEA	A	CHOCORUA	D	CLAYTON	B	COHOCTAH	B/D	COMETA	D
CHEMAWA	B	CHOICE	D	CLE ELUM	C	COHOE	B	COMFORT	D
CHEN	D	CHOOP	D	CLEAR LAKE	D	COILS	C	COMFREY	B/D
CHENA	A	CHOPTIE	D	CLEARBROOK	D	COIT	D	COMITAS	A
CHENANGO	A	CHORALMONT	B	CLEARFIELD	C	COKEDALE	C	COMLY	C
CHENAULT	B	CHOSKA	B	CLEARFORK	D	COKEL	B	COMMERCE	C
CHENEGA	A	CHOTEAU	C	CLEARWATER	D	COKER	D	COMO	A
CHENEY	B	CHRIS	C	CLEAVAGE	D	COKESBURY	D	COMOBABI	D
CHENNEBY	C	CHRISMAN	D	CLEAVER	D	COKEVILLE	B	COMODORE	D
CHENOWETH	B	CHRISTIAN	C	CLEBIT	D	COLAND	B/D	COMORO	B
CHEQUEST	C	CHRISTIANA	C	CLEGG	B	COLBAR	C	COMPASS	B
CHERIONI	D	CHRISTIANBURG	C	CLENAN	B	COLBERT	D	COMPTCHE	B
CHEROKEE	D	CHRISTINE	D	CLEMS	B	COLBURN	C	COMSTOCK	C
CHERRY	C	CHRISTY	C	CLENVILLE	B	COLBY	B	COMUS	B
CHERRY SPRING	C	CHROME	C	CLENDENEN	D	COLDCREEK	B	CONA	C
CHERRYHILL	B	CHRYSLER	C	CLEORA	B	COLE	C	CONABY	B/D
CHERUM	B	CHUALAR	B	CLERF	C	COLE, MODERATELY	C	CONALB	B
CHESAW	A	CHUBBS	C	CLERGERN	B	WET		CONANT	C
CHESHIRE	B	CHUCKANUT	B	CLERMONT	D	COLE, DRAINED	B	CONASAUGA	C
CHESHNINA	C	CHUCKAWALLA	B	CLEVELAND	C	COLEMAN	C	CONATA	D
CHESTATEE	B	CHUCKLES	B	CLEVERLY	B	COLEMANTOWN	C/D	CONBOY	D
CHESTER	B	CHUGCREEK	C	CLEVES	B	COLESTINE	C	CONCEPCION	D
CHESTERTON	D	CHUGTER	B	CLICK	A	COLFAX	C	CONCHAS	C
CHESTONIA	D	CHULITNA	B	CLIFFDELL	B	COLIBRO	B	CONCHO	C
CHESUNCOOK	C	CHUNSTICK	C	CLIFFDOWN	B	COLINAS	B	CONCONULLY	B
CHETCO	D	CHUPADERA	C	CLIFFHOUSE	C	COLITA	D	CONCORD	D
CHETEK	B	CHURCH	D	CLIFFORD	C	COLLANER	C	CONDA	D
CHETWYND	B	CHURCHILL	D	CLIFTERSON	B	COLLARD	B	CONDIE	B
CHEVELON	C	CHURCHVILLE	D	CLIFTON	B	COLLBRAN	D	CONDIT	D
CHEVIOT	B	CHURN	B	CLIFTY	B	COLLBRAN, COBBLY	C	CONDON	C
CHEWACLA	C	CHUSKA	D	CLIMARA	D	COLLEGEDALE	C	CONE	A
CHEWELAH	C	CHUTE	A	CLIMAX	D	COLLEGIATE	D	CONEJO	B
CHEYENNE	B	CIALES	D	CLIME	C	COLLEGIATE,	C	CONEJO, BEDROCK	B
CHIA	D	CIBEQUE	B	CLINT	C	FLOODED		SUBSTRATUM	
CHIARA	D	CIBO	D	CLINTON	B	COLLETT		CONEJO, GRAVELLY	C
CHICANE	C	CIBOLA	B	CLODINE	D	COLLETT, DRAINED	C	SUBSTRATUM	
CHICKAHOMINY	D	CIDRAL	C	CLONTARF	B	COLLIER	A	CONESTOGA	B
CHICKASHA	B	CIENEBA	C	CLOQUALLUM	C	COLLINGTON	B	CONESUS	B
CHICKREEK	D	CIENO	D	CLOQUATO	B	COLLINS	C	CONETOE	A
CHIEFLAND	B	CIMARRON	C	CLOQUET	B	COLLINSTON	B	CONGAREE	B
CHIGLEY	C	CINCINNATI	C	CLOUD PEAK	C	COLLINSVILLE	C	CONGER	C
CHIKAMIN	C	CINCO	A	CLOUD RIM	B	COLLINWOOD	C	CONGER, COBBLY	D
CHILAD	C	CINDERHURST	D	CLOUDCROFT	D	COLMA	B	SUBSTRATUM	

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CONGLE	B	CORRALITOS, SILTY	B	COYATA	C	CROSSTELL	D	CUTHBERT, STONY	C
CONI	D	SUBSTRATUM		COYNE	B	CROSSVILLE	B	CUTHBERT, GRADED	D
CONIC	C	CORRECO	C	COZAD	B	CROSWELL	A	CUTOFF	C
CONKLIN	B	CORRIGAN	D	COZBERG	B	CROT	D	CUTSHIN	B
CONLEN	B	CORSON	C	COZTUR	D	CROTON	D	CUTZ	D
CONLEY	C	CORTA	D	CRABTREE	B	CROUCH	B	CUYON	A
CONNEAUT	C	CORTADA	B	CRADDOCK	B	CROW	C	CYAN	B
CONNEL	B	CORTEZ	D	CRADLEBAUGH	D	CROW CREEK	B	CYCLONE	B/D
CONNERTON	B	CORTINA	B	CRAFT	B	CROW HILL	C	CYLINDER	B
CONOSTA	C	CORTINA, STONY	A	CRAFTON	C	CROWCAMP	D	CYMRIC	D
CONOTTON	B	CORTINA, FLOODED	A	CRAGGEY	D	CROWFLATS	B	CYNTHIANA	D
CONOVER	C	CORTINA, THIN	A	CRAGO	B	CROWFOOT	B	CYPHER	D
CONOWINGO	C	SURFACE		CRAGOLA	D	CROWHEART	C	CYRIL	B
CONRAD	A/D	CORTINA, PROTECTED	B	CRAGOSEN	D	CROWLEY	D	CZAR	B
CONROE	B	CORUNNA	B/D	CRAIG	B	CROWNEST	D	DABNEY	A
CONSEJO	C	CORWIN	B	CRAIGMILE	B/D	CROWSHAW	B	DABOB	B
CONSER	D	CORWITH	B	CRAIGSVILLE	B	CROYDON	B	DACKER	C
CONSTABLE	A	CORY	C	CRAMER	D	CROZIER	C	DACONO	C
CONSTANCIA	D	CORYDON	D	CRAMONT	C	CRUCES	D	DACORE	B
CONSUMO	B	COSAD	C	CRANE	B	CRUCKTON	B	DACOSTA	D
CONTEE	D	COSEY	B	CRANECREEK	C	CRUICKSHANK	A/D	DADE	A
CONTIDE	B	COSH	C	CRANFILL	B	CRUISER	B	DAGAN	B
CONTINE	C	COSHOCTON	C	CRANNLER	B	CRUMARINE	B	DAGFLAT	C
CONTINENTAL	C	COSKI	B	CRANSTON	B	CRUME	B	DAGLUM	D
CONTO	B	COSTILLA	A	CRARY	C	CRUMP	B/D	DAGOR	B
CONTRA COSTA	C	COSUMNES	C	CRASH	B	CRUMP, DRAINED	C	DAGUAD	C
CONVENT	C	COTACO	C	CRATER LAKE	B	CRUNKER	B	DAGUEY	B
CODERS	B	COTANT	D	CRATERMO	C	CRUTCH	C	DAHLQUIST	B
COOK	D	COTATI	C	CRAVEN	C	CRUTCHER	C	DAICK	D
COOKPORT	C	COTEAU	C	CRAWFORD	D	CRYSTAL LAKE	B	DAIGLE	C
COOLBRITH	C	COTHA	C	CREAL	C	CRYSTAL SPRINGS	D	DAILEY	A
COOLIDGE	B	COTITO	B	CREASEY	C/D	CRYSTALBUTTE	B	DAINT	B
COOLVILLE	C	COTO	B	CREDO	B	CUBA	B	DAKOTA	B
COOMBS	B	COTOPAXI	A	CREED	C	CUBERANT	B	DALBO	B
COONEY	B	COTT	B	CREEDMOOR	C	CUCHILLAS	C	DALBY	D
COOPER	B	COTTER	B	CREEMON	B	CUDAHY	D	DALCAN	C
COOSAW	D	COTTERAL	B	CREIGHTON	B	CUDAHY, DRAINED	C	DALCO	D
COOSBAY	B	COTTONEVA	C	CRELDON	C	CUDAHY, VERY	D	DALE	B
COOTER	C	COTTONTHOMAS	B	CREN	B	POORLY DRAINED		DALEVILLE	D
COPAKE	B	COTTONWOOD	C	CRESBARD	C	CUDDEBACK	C	DALHART	B
COPALIS	B	COTTRELL	C	CRESCO	C	CUERO	B	DALIAN	B
COPASTON	D	COTULLA	D	CRESKEN	B	CUESTA	C	DALIG	B
COPELAND	B/D	COUCH	D	CRESPIN	C	CUEVA	D	DALKENA	B
COPELAND,	D	COUGARBAY	D	CREST	C	CUEVITAS	D	DALLAM	B
DEPRESSIONAL		COUGHANOUR	C	CRESTLINE	B	CUEVOLAND	B	DALLARDSVILLE	C
COPEMAN	B	COULSTONE	B	CRESTMAN	D	CULBERTSON	B	DALLESPORT	B
COPENHAGEN	D	COULTER	B	CRESTVALE	C	CULDESAC	B	DALTON	C
COPITA	B	COUNCELOR	B	CRETE	C	CULLABY	D	DALUPE	B
COPPER RIVER	D	COUNCIL	B	CREVA	D	CULLEN	C	DALZELL	C
COPPEREID	D	COUNTRYMAN	C	CREVASSE	A	CULLEOKA	B	DAMASCUS	B/D
COPPERTON	B	COUNTS	D	CREWS	D	CULP	C	DAMEWOOD	C
COPPOCK	B	COUPEE	B	CRIDER	B	CULPEPER	C	DAMLUIS	C
COPSEY	D	COUPEVILLE	C	CRIMS	D	CULTUS	B	DAMON	D
COQUILLE	D	COURT	B	CRINKER	C	CULVING	C	DANA	B
CORA	D	COURTHOUSE	D	CRIPPIN	B	CUMBERLAND	B	DANCY	B/D
CORAL	C	COURTLAND	B	CRISFIELD	B	CUMBRES	C	DANCY, STONY	D
CORBETT	B	COURTNEY	D	CRISTO	C	CUMLEY	C	DANDREA	C
CORBIN	B	COURTROCK	B	CRISTOBAL	B	CUMMINGS	D	DANDRIDGE	D
CORCEGA	C	COUSE	C	CRITCHELL	B	CUNDICK	D	DANGBERG	D
CORDELL	D	COUSHATTA	B	CRITTENDEN	B	CUNDIYO	B	DANIA	B/D
CORDES	B	COUTIS	B	CROATAN	D	CUNNINGHAM	C	DANJER	D
CORDESTON	B	COVE	D	CROCKER	A	CUPCO	C	DANKO	D
CORDOVA	C/D	COVELAND	C	CROCKETT	D	CUPOLA	B	DANLEY	C
CORDY	B	COVELLO	C	CROESUS	C	CUPPER	B	DANNEMORA	D
CORIFF	B/D	COVERT	A	CROFTON	B	CUPPLES	C	DANSKIN	B
CORINTH	C	COVEYTOWN	C	CROGHAN	B	CURABITH	A	DANT	D
CORKSTONE	D	COVILLE	B	CROMWELL	A	CURANT	B	DANVERS	C
CORLENA	A	COVING	C	CRONKHITE	C	CURDLI	C	DANVILLE	C
CORLETT	A	COVINGTON	D	CRONKS	C	CURECANTI	B	DAPHNEDALE	C
CORLEY	B/D	COWAN	A	CROOKED	C	CURHOLLOW	D	DARBY	B
CORMANT	A/D	COWARTS	C	CROOKED CREEK	D	CURRAN	C	DARCO	A
CORNELIA	A	COWCO	B	CROOKED CREEK,	C	CURRIER	A	DARDANELLE	B
CORNELIUS	C	COWDEN	D	DRAINED		CURRITUCK	D	DARDEN	A
CORNHILL	B	COWDREY	C	CROOKED CREEK,	D	CURTIN	D	DARDOOW	B
CORNICK	D	COWEEMAN	C	RARELY FLOODED		CURTIS CREEK	D	DARE	D
CORNING	D	COWERS	B	CROOKED CREEK,	D	CURTIS SIDING	A	DARFUR	B/D
CORNISH	C	COWETA	C	VERY POORLY		CURTISTOWN	B	DARGOL	D
CORNUTT	C	COWGIL	B	DRAINED		CUSHENBURY	B	DARIEN	C
CORNVILLE	B	COWHORN	B	CROOKED CREEK, LOW	D	CUSHING	B	DARKBULL	B
COROLLA	D	COWICHE	B	PRECIPITATION		CUSHMAN	C	DARL	C
CORONA	B	COWOOD	D	CROOKSTON	B	CUSHOOL	C	DARLAND	B
CORONACA	B	COWSLY	C	CROOM	C	CUSICK	C	DARLING	B
COROZAL	C	COWTON	C	CROPLEY	D	CUSTCO	B	DARMSTADT	D
COROZO	A	COX	D	CROQUIB	D	CUSTER	D	DARNELL	C
CORPENING	D	COXVILLE	D	CROSBY	C	CUSTER, DRAINED	C	DARNEN	B
CORRAL	C	COXWELL	C	CROSIER	C	CUTAWAY	B	DAROW	C
CORRALITOS	A	COY	D	CROSS	D	CUTHAND	B	DARR	B
		COYANOSA	D	CROSSPLAIN	C/D	CUTHBERT	C	DARRET	C

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MODIFIERS SHOWN, E.G., BEDROCK SUBSTRATUM, REFER TO A SPECIFIC SOIL SERIES PHASE FOUND IN SOIL MAP LEGEND.

TABLE 7.1--HYDROLOGIC GROUPS OF THE SOILS OF THE UNITED STATES

DARROCH	C	DEFORD	A/D	DENVER	C	DILLWYN	A	DONEY	C
DARROCH, TILL	B	DEGARMO	D	DEODAR	D	DILMAN	C	DONICA	A
SUBSTRATUM		DEGNER	C	DEPALT	D	DILTON	D	DONIPHAN	B
DARROCH, BEDROCK	C	DEGOLA	B	DEPCOR	B	DILTS	D	DONKEHILL	D
SUBSTRATUM		DEGRAND	B	DEPORT	D	DIMAL	C	DONLONTON	C
DARROUZETT	C	DEGREY	D	DEPUTY	C	DIMMICK	D	DONNA	D
DARSIL	C	DEHANA	B	DERA	B	DIMO	B	DONNAN	C
DARST	C	DEHART	B	DERALLO	B	DIMYAW	C	DONNARDO	B
DART	A	DEHLINGER	B	DERB	C	DINA	C	DONNEL	B
DARVEY	B	DEJARNET	B	DERECHO	C	DINES	B	DONNELLY	A
DARWIN	D	DEKALB	C	DERINDA	C	DINEVO	B	DONNER	C
DASHER	D	DEKOVEN	D	DERLY	D	DINGLE	C	DONNYBROOK	D
DASSEL	B/D	DEL REY	C	DEROUX	C	DINGLISHNA	D	DOODLELINK	B
DAST	B	DELA	B	DERRICK	B	DINKELMAN	B	DOOLEY	C
DATELAND	B	DELANCO	C	DES MOINES	C	DINKELS	B	DOONE	B
DATEMAN	C	DELAND	A	DES MOINES, DRY	B	DINNEN	B	DOOR	B
DATIL	B	DELANEY	A	DESAN	A	DINSDALE	B	DORA	B/D
DATINO	D	DELANO	B	DESART	C	DINUBA	C	DORAN	C
DATINO, STONY	B	DELASSUS	C	DESATOYA	C	DINWOODY	B	DORB	C
DATWYLER	C	DELCOMB	D	DESCALABRADO	D	DIOBSUD	B	DORCHESTER	B
DAULTON	D	DELOOTA	D	DESCHELL	B	DIOXICE	B	DORERTON	B
DAVEY	B	DELECO	C	DESCHUTES	C	DIPMAN	D	DORMONT	C
DAVIDELL	B	DELENA	D	DESCOT	B	DIPSEA	B	DORNA	B
DAVIDSON	B	DELEON	C	DESEED	C	DIQUE	B	DOROSHIN	D
DAVIS	B	DELETTE	C	DESERET	C	DIREGO	D	DOROTHEA	C
DAVISON	B	DELFINA	B	DESHA	D	DISABEL	D	DOROVAN	D
DAVTONE	B	DELFT	B/D	DESHLER	C	DISAUTEL	B	DORRANCE	A
DAWES	C	DELGADO	D	DESKAMP	C	DISCO	B	DORS	B
DAWHOO	B/D	DELHI	A	DESMET	B	DISHNER	D	DORSET	B
DAWSON	A/D	DELICIAS	B	DESPAIN	B	DISTERHEFF	C	DOSAMIGOS	D
DAWTONIA	B	DELKS	C/D	DESTAZO	B	DISTON	C	DOSPALOS	D
DAXTY	B	DELL	C	DESTER	B	DITCHCAMP	C	DOSS	C
DAY	D	DELLEKER	B	DETER	C	DITHOD	C	DOSSMAN	B
DAYBELL	A	DELLO, SALINE	C	DETOUR	B	DITNEY	C	DOTARD	B
DAYSCHOOL	B	DELLO, GRAVELLY	D	DETRA	B	DIVERS	B	DOTEN	D
DAYTON	D	SUBSTRATUM, WET		DETRITAL	B	DIVIDE	B	DOTHAN	B
DAYTONA	B	DELLO,	A	DETROIT	C	DIVOT	C	DOTLAKE	D
DAYVILLE	C	SALINE-ALKALI		DEUNAH	D	DIX	A	DOTSERO	B
DAZE	D	DELLO, MODERATELY	C	DEV	A	DIXALETA	D	DOTTA	B
DE BACA	B	WET		DEVADA	D	DIXBORO	B	DOTY	B
DE MASTERS	B	DELLO, CLAY	B	DEVEN	D	DIXIE	C	DOUCETTE	B
DE PERE	C	SUBSTRATUM		DEVILS	D	DIXMONT	C	DOUDLE	B
DEACON	B	DELLROSE	B	DEVINE	C	DIXONVILLE	C	DOUDS	B
DEADMAN	B	DELLS	C	DEVISADERO	C	DIYOU	C	DOUGAL	D
DEADWOOD	D	DELMA	C	DEVOE	D	DOAK	B	DOUGAN	B
DEANA	D	DELNITA	C	DEVOIGNES	D	DOAK, MODERATELY	C	DOUGCLIFF	D
DEAN	B	DELMONT	B	DEVOL	B	ALKALI		DOUGHERTY	A
DEANDALE	D	DELNORTE	C	DEVORE	B	DOBBINS	C	DOUGHTY	B
DEARBORN	B	DELORO	C	DEVOY	D	DOBBS	B	DOUGLAS	B
DEARYTON	C	DELOSS	B/D	DEVRIES	C	DOBEL	D	DOUGVILLE	B
DEATMAN	C	DELP	A	DEWAR	D	DOBENT	C	DOUHIDE	D
DEAVER	C	DELPHI	B	DEWEY	B	DOBROW	B/D	DOURO	B
DEBENGER	C	DELPHILL	C	DEWEYVILLE	D	DOBY	D	DOVER	B
DEBONE	D	DELPIEDRA	D	DEWVILLE	B	DOCAS	B	DOVRAY	C/D
DEBORAH	D	DELPLAIN	D	DEXTER	B	DOCDEE	D	DOW	B
DEBUTE	C	DELPOINT	C	DIA	C	DOCENA	C	DOWAGIAC	B
DECAN	C	DELRADORE	D	DIA, WET, SALINE	D	DOCKERY	C	DOWDE	B
DECANTEL	D	DELRAY	B/D	DIA, SALINE	C	DOCT	C	DOWELLTON	D
DECATHON	C	DELRAY,	D	DIA, WET	D	DODES	B	DOWNATA	D
DECATUR	B	DEPRESSIONAL		DIA, FLOODED	D	DODGE	B	DOWNER	B
DECCA	B	DELRAY, FLOODED	B/D	DIABLO	D	DODGEVILLE	B	DOWNEY	B
DECHEL	D	DELRIDGE	B	DIAMANTE	B	DODSON	C	DOWNEYVILLE	D
DECKER	C	DELSON	C	DIAMOND	D	DOGER	A	DOWNS	B
DECKERVILLE	D	DELTON	B	DIAMOND SPRINGS	C	DOGUE	C	DOYCE	B
DECKERVILLE,	C	DELWIN	A	DIAMONDVILLE	C	DOLAND	B	DOYCE, LOAMY	C
DRAINED		DELYNDIA	A	DIANEV	C	DOLBEE	C	SUBSTRATUM	
DECLO	B	DEMAR	D	DIANOLA	D	DOLEKEI	B	DOYCE, MODERATELY	C
DECOLNEY	B	DEMAST	B	DIATEE	B	DOLEN	B	WET	
DECORDOVA	B	DEMENT	C	DIBBLE	C	DOLES	C	DOYLESTOWN	D
DECRAM	C	DEMING	B	DIBOLL	C	DOLLAR	C	DOYN	D
DECROSS	B	DEMKY	D	DICK	A	DOLLARD	C	DRA	C
DECY	C	DEMNER	B	DICKERSON	D	DOLLARHIDE	D	DRAGE	B
DEE	C	DEMONA	C	DICKEY	B	DOLLYCLARK	C	DRAGOON	B
DEEFAN	D	DEMONTREVILLE	B	DICKINSON	B	DOLPH	C	DRAGSTON	C
DEEMER	B	DEMOPOLIS	C	DICKINSON, MAP<25	B	DOME	B	DRAKE	B
DEEPEEK	D	DEMOPOLIS, COBBLY	D	DICKINSON, TILL	A	DOMELL	B	DRALL	B
DEEPWATER	B	DEMPSEY	B	SUBSTRATUM		DOMERIE	B	DRANYON	B
DEER CREEK	C	DEMPSTER	B	DICKMAN	A	DOMINGUEZ	C	DRAPER	C
DEER PARK	A	DENAY	B	DICKSON	C	DOMINIC	B	DRAX	B
DEERFIELD	B	DENHAWKEN	D	DIEHLSTADT	C	DOMINO	C	DRAX, WET	C
DEERFORD	D	DENMAN	C	DIETRICH	C	DOMINSON	A	DREDGE	B
DEERHORN	C	DENMARK	D	DIGBY	B	DOMO	B	DRESDEN	B
DEERLODGE	C	DENNIS	C	DIGGER	C	DONA ANA	B	DRESSLER	C
DEERTON	A	DENNOT	B	DIGHTON	B	DONAHUE	C	DREWING	D
DEERTRAIL	C	DENNY	D	DIGIORGIO	B	DONALD	C	DREWS	B
DEERWOOD	B/D	DENROCK	D	DILL	B	DONALDSON	B	DREXEL	B
DEETZ	A	DENTON	D	DILLARD	C	DONAVAN	B	DRIFTWOOD	C/D
DEFIANCE	D	DENURE	B	DILLEY	B	DONERAIL	C	DRIGGS	B

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DRISCOLL	C
DRIT	B
DRIVER	C
DROEM	C
DRUM	C
DRUMMER	B/D
DRUMMOND	D
DRURY	B
DRY CREEK	C
DRYADINE	C
DRYBURG	B
DRYDEN	B
DRYN	C
DU PAGE	B
DUANE	B
DUART	C
DUBAKELLA	C
DUBBS	B
DUBLON	B
DUBOIS	C
DUBUQUE	B
DUCHESNE	B
DUCKHILL	D
DUCKREE	B
DUCKSTON	A/D
DUCO	D
DUDA	A
DUDLEY	D
DUEL	A
DUELM	A
DUETTE	A
DUFF	B
DUFFAU	B
DUFFER	B/D
DUFFER, DRAINED	C
DUFFER, FLOODED	B/D
DUFFIELD	B
DUFFSON	B
DUFORT	B
DUFUR	B
DUGGINS	C
DUGOUT	D
DUGWAY	C
DUKES	A
DULAC	C
DULCE	D
DULUTH	B
DUMAS	B
DUMONT	B
DUN GLEN	B
DUNBAR	D
DUNBARTON	D
DUNBRIDGE	B
DUNCAN	D
DUNCANNON	B
DUNCKLEY	B
DUNCOM	D
DUNDAS	B/D
DUNDAY	A
DUNDEE	C
DUNELLEN	B
DUNFORD	C
DUNGENESS	B
DUNKIRK	B
DUNLATOP	B
DUNMORE	B
DUNN	A
DUNNING	D
DUNNVILLE	B
DUNOIR	B
DUNPHY	C
DUNPHY, DRAINED	B
DUNTON	C
DUNUL	A
DUPEE	C
DUPLIN	C
DUPO	C
DUPONT	D
DUPREE	D
DURADOS	A
DURALDE	C
DURAND	B
DURANGO	B
DURANT	D
DURBIN	D
DURELLE	B
DURFEE	C
DURHAM	B

DURKEE	C
DUROC	B
DURRSTEIN	D
DURST	C
DUSLER	C
DUSTON	A
DUTCHESS	B
DUTEK	A
DUTTON	D
DUVAL	B
DUXBURY	A
DUZEL	C
DWIGHT	D
DWORSHAK	B
DWYER	A
DYE	D
DYKE	B
DYRENG	D
EACHUS	B
EACHUSTON	A/D
EAGAR	B
EAGLECONE	B
EAGLEVILLE	D
EAKIN	B
EALY	B
EAPA	B
EARCREE	B
EARLE	D
EARLMONT	D
EARLMONT, DRAINED	C
EARP	B
EARSNAN	D
EASLEY	C
EAST FORK	C
EAST LAKE	A
EASTABLE	B
EASTCAN	B
EASTGATE	B
EASTLAND	B
EASTON	D
EASTONVILLE	B
EASTPORT	A
EASTWELL	D
EATON	D
EAUGALLIE	B/D
EAUPLEINE	B
EBA	C
EBAL	B
EBBERT	C/D
EBIC	C
EBODA	B
EBON	C
EBRO	D
ECCLES	B
ECHARD	D
ECHAW	B
ECHEMOOR	C
ECKERT	D
ECKLEY	B
ECKMAN	B
ECKRANT	D
ECKVOLL	B
ECOLA	C
ECONFINA	A
ECTOR	D
EDALGO	C
EDDINGS	B
EDDS	B
EDDY	C
EDEN	C
EDENBOWER	D
EDENTON	C
EDFRO	D
EDGAR	B
EDGEHILL	C
EDGELEY	C
EDGEMONT	B
EDGEWATER	D
EDGEWICK	C
EDGINGTON	C/D
EDINA	D
EDINBURG	C
EDISTO	C
EDLOE	B
EDMINSTER	D
EDMONDS	D
EDMORE	D
EDMUND	D

EDMUNDSTON	B
EDNA	D
EDNEYTOWN	B
EDNEYVILLE	B
EDOM	C
EDROY	D
EDSON	C
EDWARDS	B/D
EEL	B
EEP	C
EFFIE	C
EFFINGTON	D
EGAM	C
EGAN	B
EGAS	D
EGBERT	D
EGBERT, MODERATELY WET	C
EGBERT, DRAINED	C
EGBERT, SANDY SUBSTRATUM	C
EGELAND	B
EGINBENCH	C
EGYPT	D
EICKS	C
EIGHTLAR	D
EIGHTMILE	D
EILERTSEN	B
EITZEN	B
EKAH	C
EKALAKA	B
EKRUB	D
EL DARA	B
EL PECO	C
EL RANCHO	B
EL SOLYO	C
ELANDCO	B
ELBA	C
ELBAVILLE	B
ELBERT	D
ELBETH	B
ELBON	B
ELBURN	B
ELBUTTE	D
ELCO	B
ELD	B
ELDEAN	B
ELDER	B
ELDER, GRAVELLY SUBSTRATUM, FLOODED	A
ELDER, FLOODED	B
ELDER, GRAVELLY	B
ELDER, GRAVELLY SUBSTRATUM	A
ELDER HOLLOW	D
ELDERON	B
ELDERON, STONY	A
ELDGIN	B
ELDON	B
ELDORADO	B
ELDRIDGE	C
ELECTRA	C
ELEROY	B
ELEVA	B
ELFRIDA	B
ELGEE	A
ELIJAH	C
ELINDIO	C
ELIOAK	C
ELIZA	D
ELK	B
ELK HOLLOW	B
ELK MOUNTAIN	B
ELKADER	B
ELKCREEK	C
ELKHART	B
ELKHORN	B
ELKINS	D
ELKINSVILLE	B
ELKMOUND	D
ELKNER	B
ELKOL	D
ELKSEL	C
ELKTON	C/D
ELLABELLE	D
ELLEDGE	C
ELLEN	B

ELLETT	D
ELLIBER	A
ELLICOTT	A
ELLINGTON	B
ELLIOTT	C
ELLIOTTSVILLE	B
ELLIS	D
ELLISFORDE	B
ELLISVILLE	B
ELLOAM	D
ELLSWORTH	C
ELLUM	C
ELLZEY	B/D
ELM LAKE	A/D
ELMDALE	B
ELMENDORF	D
ELMINA	C
ELMIRA	A
ELMONT	B
ELMORE	B
ELMRIDGE	C
ELMVILLE	B
ELMWOOD	C
ELNIDO	C
ELNORA	B
ELO	B
ELOCHOMAN	B
ELOIKA	B
ELOMA	C
ELPAN	D
ELPEDRO	B
ELRED	B/D
ELRIN	B
ELROD	D
ELROSE	B
ELS	A
ELSAH	B
ELSINBORO	B
ELSMERE	A
ELSTON	B
ELTREE	B
ELTSAC	D
ELVADA	B
ELVE	B
ELVEDERE	C
ELVERS	B/D
ELVIRA	B/D
ELWELL	C
ELWHA	C
ELWOOD	C
ELY	B
ELYSIAN	B
ELZINGA	B
EMBAL	B
EMBARGO	C
EMBDEN	B
EMBERTON	C
EMBLEM	B
EMBRY	B
EMBUDO	B
EMDENT	D
EMDENT, BEDROCK SUBSTRATUM, DRAINED	C
EMDENT, BEDROCK SUBSTRATUM	D
EMDENT, DRAINED	C
EMERALD	B
EMERALDA	D
EMERSON	B
EMIGRANT	C
EMIGRATION	D
EMILY	B
EMMA	C
EMMERT	A
EMMET	B
EMORY	B
EMOT	B
EMPEDRADO	B
EMPEYVILLE	C
EMPIRE	B
EMPORIA	C
EMRICK	B
ENBAR	B
ENCAMPMENT	B
ENCIERRO	D
ENCINA	B
ENDCAV	C

ENDERS	C
ENDERSBY	B
ENDICOTT	C
ENDLICH	B
ENDSAW	C
ENERGY	B
ENET	B
ENFIELD	B
ENGELHARD	B/D
ENGLE	B
ENGLEWOOD	C
ENKO	C
ENLOE	D
ENNING	D
ENNIS	B
ENOCH	C
ENOCHVILLE	D
ENOCHVILLE, DRAINED	C
ENON	C
ENOREE	D
ENOS	C
ENOSBURG	D
ENSENADA	B
ENSIGN	D
ENSLEY	B/D
ENSTROM	B
ENTENTE	B
ENTERPRISE	B
ENTIAT	D
ENTMOOT	C
EOJ	C
EPHRAIM	C
EPHRATA	B
EPIKOM	D
EPLEY	C
EPOKE	B
EPOUFETTE	B/D
EPPING	D
EPSIE	D
ERA	B
ERAM	C
ERAMOSH	D
ERBER	C
ERCAN	B
ERD	D
ERICSON	B
ERIE	C
ERIN	B
ERNEN	D
ERNEST	C
ERNO	B
ERRAMOUSPE	C
ERVIDE	C
ESCABOSA	C
ESCALANTE	B
ESCAMBIA	C
ESCANO	C
ESCONDIDO	C
ESHAMY	B
ESMERALDA	B
ESMOND	B
ESPELIE	B/D
ESPIL	D
ESPINAL	A
ESPINOSA	B
ESPLIN	D
ESPY	C
ESQUATZEL	B
ESRO	D
ESS	B
ESSAL	B
ESSEN	C
ESSEX	C
ESSEXVILLE	A/D
ESTACADO	B
ESTACION	B
ESTATE	C
ESTELLINE	B
ESTER	D
ESTERO	D
ESTHERVILLE	B
ESTIVE	B
ESTO	B
ESTRELLA	B
ETACH	B
ETCHEN	C
ETELKA	C

NOTES: TWO HYDROLOGIC SOIL GROUPS SUCH AS B/C INDICATES THE DRAINED/UNDRAINED SITUATION.
MODIFIERS SHOWN, E.G., BEDROCK SUBSTRATUM, REFER TO A SPECIFIC SOIL SERIES PHASE FOUND IN SOIL MAP LEGEND.

TABLE 7.1--HYDROLOGIC GROUPS OF THE SOILS OF THE UNITED STATES

ETHAN	B	FAIRWAY	C	FENWOOD	B	FLOER	D	FOXWORTH	A
ETHELMAN	B	FAIRYDELL	C	FERA	C	FLOKE	D	FRADDLE	B
ETHETE	B	FAJAROO	C	FERDELFORO	C	FLOM	B/D	FRAILEY	B
ETHETE, SALINE	C	FALAYA	O	FEROINAND	C	FLOMATON	A	FRAILTON	D
ETHRIDGE	C	FALBA	D	FERGUS	B	FLOMOT	B	FRAM	B
ETIL	A	FALCON	D	FERN CLIFF	B	FLORALA	C	FRANCIS	A
ETOE	B	FALFA	C	FERNANOO	B	FLORENCE	C	FRANCITAS	D
ETOILE	D	FALFURRIAS	A	FERNHAVEN	B	FLORESVILLE	C	FRANDSEN	B
ETOWAH	B	FALK	C	FERNLEY	C	FLORIDANA	B/D	FRANKFORT	C
ETOWN	C	FALKIRK	B	FERNPOINT	B	FLORIN	C	FRANKIRK	C
ETSEL	O	FALKNER	C	FERNWOOD	B	FLORISSANT	C	FRANKLIN	B
ETTA	B	FALLBROOK	B	FERRELO	B	FLORITA	B	FRANKSTOWN	B
ETTER	B	FALLCREEK	C	FERRIS	O	FLOTAG	B	FRANKTOWN	D
ETTRICK	B/O	FALLERT	B	FERROBURRO	D	FLOWELL	C	FRANKVILLE	B
EUBANKS	B	FALLON	C	FERRON	O	FLOWEREE	B	FRATERNIDAD	D
EUCLIO	C	FALLSAM	D	FERTALINE	D	FLOYD	B	FRAVAL	C
EUDORA	B	FALLSINGTON	B/D	FERTEG	B	FLUETSCH	B	FRAZER	C
EUER	B	FALOMA	B/D	FESTINA	B	FLUGLE	B	FRAZERTON	B
EUFAULA	A	FALULA	D	FETT	D	FLUVANNA	C	FRED	C
EUHARLEE	C	FANAL	B	FETTIC	D	FLYBOW	D	FREDENSBORG	C
EULONIA	C	FANOANGLE	C	FETZER	C	FLYGARE	B	FREOERICK	B
EUNOLA	C	FANDOW	D	FIANDER	D	FOARD	D	FREOON	C
EUREKA	D	FANG	B	FIANOER, DRAINED	C	FOEHLIN	B	FREDONIA	C
EUSBIO	C	FANNIN	B	FIOALGO	C	FOLA	B	FREDONYER	C
EUSTIS	A	FANNO	C	FIDDLER	C	FOLDAHL	B	FREEBURG	C
EUTAW	O	FANTZ	C	FIDDLETOWN	B	FOLEY	D	FREECE	D
EVADALE	O	FANU	B	FIDDYMENT	D	FOLLET	D	FREEDOM	C
EVANGELINE	C	FARAWAY	D	FIELD	B	FOMSENG	C	FREEHOLD	B
EVANS	B	FARB	D	FIELDING	B	FONDA	D	FREEMAN	C
EVANSTON	B	FARBER	B	FIELDON	B/D	FONDIS	C	FREEMANVILLE	B
EVANSVILLE	B/O	FARGO	D	FIFER	D	FONNER	B	FREEON	B
EVANT	D	FARISITA	D	FILDERT	C	FONTANA	B	FREER	C
EVARD	B	FARLAND	B	FILION	D	FONTREEN	B	FREEST	C
EVARO	B	FARLOW	C	FILLMORE	D	FOPIANO	D	FREESTONE	C
EVART	O	FARMINGTON	C	FINCASTLE	C	FORADA	B/O	FREETOWN	D
EVENDALE	C	FARMSWORTH	D	FINCH	C	FORAKER	D	FREEWATER	B
EVERETT	A	FARMTON	D	FINCHFORD	A	FORBES	C	FREEZENER	C
EVERETT, STONY	A	FARNHAM	B	FINDOUT	D	FORBING	D	FREEZEOUT	C
EVERETT, HARO	B	FARNHAMTON	C	FINGAL	C	FORD	D	FRELSBURG	D
SUBSTRATUM		FARNUF	B	FINGEROCK	D	FOROICE	B	FREMONT	C
EVERGLADES	B/O	FARNUF, WET	C	FINLEY	B	FORONEY	A	FREN	B
EVERLY	B	FARNUF, GRAVELLY	B	FINNERTY	D	FORDTRAN	C	FRENCH	C
EVERMAN	C	SUBSTRATUM		FINOL	C	FORDUM	D	FRENCHCREEK	B
EVERSON	D	FARNUM	B	FINROD	C	FORDVILLE	B	FRENCHTOWN	D
EVERWHITE	C	FARRAGUT	C	FIRAOA	C	FORELAND	B	FRESHWATER	D
EVESBORO	A	FARRAR	B	FIREBALL	B	FORELLE	B	FRESNO,	D
EWA	B	FARRELL	B	FIREBOX	B	FORESMAN	B	SALINE-ALKALI	
EWA, BEDROCK	C	FARRENBURG	B	FIRESTEEL	B	FORESTBURG	A	FRESNO, THICK	C
SUBSTRATUM		FARROT	C	FIRESTONE	C	FORESTDALE	D	SOLUM	
EWALL	A	FARVA	C	FIRMAGE	B	FORESTER	C	FREWA	B
EXCELSIOR	B	FASHING	O	FIRO	D	FORESTON	C	FREZNIK	O
EXCHEQUER	O	FASKIN	B	FIROKE	B	FORK	C	FRIANA	D
EXEL	C	FATHOM	A	FIRTH	C	FORKWOOO	B	FRIANT	D
EXETER	C	FATIMA	B	FIRTH, DRAINEO	B	FORMADER	C	FRIOLO	C
EXETER, THICK	B	FATTIG	C	FISHHOOK	D	FORMAN	B	FRIENOS	D
SOLUM		FAUQUIER	C	FISHLAKE	O	FORMDALE	B	FRIENDSHIP	A
EXETTE	B	FAUSSE	D	FISHPOT	C	FORNEY	D	FRIES	D
EXIRA	B	FAVRET	C	FISK	B	FORREST	C	FRIESLAND	B
EXLINE	O	FAWIN	B	FITCHVILLE	C	FORSEER	C	FRIJOLES	B
EXPRESS	B	FAX	C	FITZGERALO	B	FORSEY	B	FRINDLE	C
EXRAY	D	FAXON	B/O	FITZHUGH	B	FORSGREN	C	FRIO	B
EXUM	C	FAYETTE	B	FIVEMILE	B	FORSYTH	A	FRIONA	C
EYAK	C	FAYETTEVILLE	B	FIVEMILE, SALINE	C	FORT COLLINS	B	FRIOTON	C
EYERBOW	C	FAYWOOD	C	FIVEOH	B	FORT MEAOE	A	FRIPP	A
EYLAU	C	FE	O	FIVES	B	FORT MOTT	A	FRISCO	B
EYOTA	A	FEATHERLEGS	B	FLAGG	B	FORTANK	C	FRIZZELL	C
EYRE	O	FEDJI	A	FLAGLER	B	FORTESCUE	C/D	FROBERG	O
FABIUS	B	FEOORA	B/O	FLAGSTAFF	O	FORTUNA	D	FRODO	D
FACEVILLE	B	FELAN	B	FLAK	C	FORTWINGATE	C	FROHMAN	C
FACEY	B	FELDA	B/D	FLAMING	A	FORVIC	C	FROLIC	B
FACTORY	C	FELOA,	O	FLANAGAN	B	FORWARD	B	FROLIC,	C
FACTORY, MOIST	B	OEPRESSIONAL		FLANDREAU	B	FOSS	B	ELEVATION<8000	
FADDIN	O	FELDA, FLOOOEO	B/D	FLANE	C	FOSSILON	O	FROLIC, FLOODEO	B
FAGAN	C	FELICITY	A	FLASHER	D	FOSSUM	A/D	FRONDORF	B
FAGASA	C	FELIPE	D	FLATHEAD	B	FOSTER	C	FRONTENAC	B
FAHEY	B	FELKER	B	FLATHORN	B	FOSTORIA	B	FRONTON	O
FAIM	C	FELLOWSHIP	D	FLATIRONS	C	FOUNTAIN	D	FROST	D
FAIRBANKS	B	FELOR	B	FLATNOSE	B	FOUR STAR	D	FRUITA	B
FAIRCHILD	C	FELT	B	FLATRON	D	FOUR STAR, DRAINED	B	FRUITHURST	C
FAIRDALE	B	FELTA	C	FLATTOP	D	FOURCHE	B	FRUITLAND	B
FAIRFAX	B	FELTHAM	B	FLAXTON	B	FOURLOG	D	FRUITLAND, WET	C
FAIRFIELO	B	FELTNER	D	FLEAK	D	FOURMILE	B	FRUITLAND, COOL	B
FAIRHAVEN	B	FELTON	B	FLEER	A/D	FOX	B	FRYE	C
FAIRLIE	D	FELTONIA	B	FLEISCHMANN	D	FOXCREEK	C	FRYEBURG	B
FAIRMOUNT	O	FENCE	B	FLEMING	C	FOXHOME	B	FT. DRUM	C
FAIRPLAY	B	FENDALL	C	FLEMINGTON	D	FOXMOUNT	C	FT. GREEN	D
FAIRPOINT	C	FENN	D	FLETCHER	B	FOXOL	O	FUBBLE	O
FAIRPORT	C	FENSTER	B	FLEX	D	FOXTON	C	FUEGO	C

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FUERA	C	GARDELLA	D	GERRARD	C	GLENDORA	A/D	GOOSE LAKE	D
FUGAWEE	B	GARDENA	B	GERST	D	GLENEDEN	D	GODSMUS	B
FUGHES	C	GARDINER	A	GESSIE	B	GLENELG	B	GORDANE	C
FULCHER	C	GARDNER'S FORK	B	GESSNER	B/D	GLENFORD	C	GORDO	B
FULDA	C/D	GARDNERVILLE	C	GESTRIN	B	GLENGARY	D	GORE	D
FULLERTON	B	GAREY	C	GETCHELL	C	GLENHALL	B	GOREEN	D
FULMER	D	GARFIELD	C	GETTYS	C	GLENHAM	B	GORGAS	D
FULMER, DRAINED	C	GARIPER	C	GETZVILLE	D	GLENMAN	B	GORGONIO	A
FULSHEAR	C	GARITA	B	GEYSEN	C	GLENMORA	C	GORHAM	B/D
FULSTONE	D	GARLAND	B	GIBBLER	C	GLENNALLEN	C	GORIN	C
FULTON	D	GARLET	A	GIBBON	B	GLENOMA	B	GORING	C
FULTS	D	GARLOCK	B	GIDEON	B/D	GLENPOOL	A	GORMAN	C
FUNTER	D	GARMON	C	GIFFORD	D	GLENRIO	D	GORSKEL	D
FUQUAY	B	GARMORE	B	GIGGER	C	GLENROSE	B	GORST	D
FURNISS	D	GARNER	D	GILA	B	GLENROSS	D	GORUS	B
FURY	C	GARNES	B	GILBERT	D	GLENSTED	D	GORZELL	B
FUSULINA	D	GARO	D	GILBY	B	GLENTON	B	GOSA	B
GAASTRA	C	GARR	D	GILCHRIST	A	GLENTON,	B	GOSHEN	B
GABALDON	B	GARRETSON	B	GILCO	B	MODERATELY WET		GOSHUTE	D
GABBVALLY	D	GARRETT	B	GILCREST	B	GLENTON, WET	C	GOSINTA	C
GABEL	C	GARRISON	B	GILEAD	C	GLENTON,	B	GOSLIN	B
GABICA	D	GARROCHALES	D	GILES	B	NONFLOODED		GOSNEY	D
GABINO	D	GARSID	C	GILFORD	B/D	GLENTON, WARM	B	GOSPER	B
GACEY	D	GARTON	C	GILFORD,	D	GLENTOSH	B	GOSPORT	C
GACHADO	D	GARVESON	B	STRATIFIED		GLENVIEW	B	GOSS	B
GACIBA	D	GARVIN	D	SUBSTRATUM		GLENVILLE	C	GOSUMI	C
GADDES	C	GARWIN	B/D	GILFORD, BEDROCK	B/D	GLOHN	C	GOTEBO	B
GADDY	A	GARZA	B	SUBSTRATUM		GLORIA	D	GOTHAM	A
GADSDEN	D	GAS CREEK	A/D	GILFORD, GRAVELLY	B/D	GLOUCESTER	A	GOTHARD	C
GAGEBY	B	GASCONADE	D	SUBSTRATUM		GLOVER	C/D	GOTHENBURG	D
GAGETOWN	B	GASIL	B	GILISPIE	D	GLYNDON	B	GOTHIC	C
GAGIL	B	GASQUET	B	GILLAND	C	GLYNN	C	GOTHO	C
GAHEE	B	GASSVILLE	C	GILLIAM	C	GLYNWOOD	C	GOULDING	D
GAIB	D	GATES	B	GILLIGAN	B	GLYPHS	B	GOVE	B
GAINES	C	GATESON	C	GILLS	C	GOBERNADOR	D	GOWEN	B
GAINESVILLE	A	GATEVIEW	B	GILLSBURG	C	GOBINE	B	GOWKER	C
GALATA	D	GATEWAY	C	GILMAN	B	GOBLE	C	GOWTON	B
GALBRETH	D	GATEWOOD	C	GILMORE	C	GOBLIN	D	GRABE	B
GALCHUTT	C	GATLIN	B	GILPAR	B	GOCHEA	B	GRABLE	B
GALE	B	GATOR	D	GILPIN	C	GODDARD	B	GRACEMONT	C
GALEN	B	GATTON	B	GILROY	C	GODDE	D	GRACEMORE	C
GALEPPI	B	GAVILAN	C	GILSTON	B	GODECKE	D	GRACEVILLE	B
GALESTOWN	A	GAVINS	D	GILT EDGE	D	GODFREY	D	GRADON	C
GALEY	B	GAVIOTA	D	GINAT	D	GODWIN	D	GRADY	D
GALILEE	C	GAY	B/D	GINI	B	GOENNER	C	GRAFEN	B
GALISTEO	C	GAYLESVILLE	D	GINLAND	D	GOESLING	B	GRAHAM	D
GALISTEO,	D	GAYLORD	C	GINNIS	C	GOESSEL	D	GRAIL	C
SALINE-ALKALI		GAYNOR	C	GINSER	C	GOGEBIC	B	GRAINOLA	D
GALLAND	D	GAYNOR, WET	D	GIRARD	D	GOL	C	GRALEY	D
GALLATIN	C	GAYVILLE	D	GIRARDOT	D	GOLCONDA	C	GRALIC	B
GALLEGOS	B	GAZELLE	D	GIRD	C	GOLD CREEK	D	GRAN	D
GALLEN	B	GAZOS	C	GIST	D	GOLDBERG	D	GRANATH	B
GALLIA	B	GEARHART	A	GITAKUP	C	GOLDENDALE	B	GRANBY	A/D
GALLINE	B	GEARY	B	GITAN	D	GOLDFINCH	D	GRANDE RONDE	D
GALLION	B	GEE	C	GIVIN	C	GOLDHILL	C	GRANDFIELD	B
GALLMAN	B	GEEBURG	C	GLACIERCREEK	A	GOLDMAN	C	GRANDPON	B
GALLUP	B	GEER	B	GLADDEN	B	GOLDMIRE	C	GRANDVIEW	C
GALOO	C/D	GEERTSEN	B	GLADEL	D	GOLDRIDGE	B	GRANER	B
GALVA	B	GEFO	A	GLADEVILLE	D	GOLDRUN	A	GRANGEMONT	C
GALVESTON	A	GEISEL	B	GLADEWATER	D	GOLDSBORO	B	GRANGEVILLE,	B
GALVEZ	C	GELKIE	B	GLADWIN	A	GOLDSTON	C	DRAINED, SLOPING	
GALVIN	D	GEM	C	GLANN	D	GOLDSTREAM	D	GRANGEVILLE,	B
GALWAY	B	GEMIO	C	GLASGOW	C	GOLDUST	C	SALINE-ALKALI	
GAMBLER	A	GEMSON	B	GLASSNER	D	GOLDVALE	B	GRANGEVILLE,	B
GAMBOA	B	GENAW	D	GLEAN	B	GOLDVEIN	C	MODERATELY WET	
GAMGEE	C	GENESEE	B	GLEASON	B	GOLDYKE	C	GRANGEVILLE, WET	C
GANCE	C	GENEVA	B	GLEN	B	GOLETA	B	GRANGEVILLE,	B
GANDO	D	GENOA	D	GLENBAR	B	GOLIAD	C	DRAINED	
GANIS	D	GENOLA	B	GLENBERG	B	GOLSUN	C	GRANGEVILLE,	B
GANNETT	D	GENTILLY	D	GLENBROOK	D	GOLTRY	A	OCCASIONALLY	
GANSNER	D	GENTRY	D	GLENCARB, WET,	C	GOLVA	B	FLOODED	
GAPCOT	D	GEOHROCK	B	SALINE		GOMERY	B	GRANILE	B
GAPO	D	GEORGETOWN	D	GLENCARB, SALINE	B	GOMEZ	B	GRANO	D
GAPO, DRAINED	C	GEORGEVILLE	B	GLENCARB, HARDPAN	B	GONVICK	B	GRANT	B
GAPPMAYER	B	GEORGIA	B	SUBSTRATUM		GOOCH	D	GRANTFORK	D
GARA	C	GEOROCK	B	GLENCARB, DRY	B	GOODING	C	GRANTHAM	D
GARBER	B	GEPFORD	D	GLENCARB,	C	GOODINGTON	C	GRANTSBURG	C
GARBO	B	GEPP	B	OCCASIONALLY		GOODLAND	B	GRANTSDALE	B
GARBUTT	B	GEPPERT	C	FLOODED		GOODLOW	B	GRANVILLE	B
GARCENO	C	GERALD	D	GLENCOE	B/D	GOODMAN	B	GRANYON	B
GARCES	D	GERBER	D	GLENCOE, PONDED	D	GOODNIGHT	A	GRANZAN	B
GARCES, MODERATELY	D	GERDRUM	C	GLENDALE	B	GOODPASTER	D	GRAPEVINE	B
WET		GERING	B	GLENDALE,	B	GOODRICH	B	GRASMERE	B
GARCES, HARD	C	GERLACH	D	SALINE-ALKALI		GOODSPRINGS	D	GRASSNA	B
SUBSTRATUM		GERLANE	B	GLENDALE, WET	C	GOODWILL	B	GRASSVAL	D
GARCIA	B	GERLE	B	GLENDALE, FLOODED	B	GOODWIN	B	GRASSVALLEY	D
GARCITAS	C	GERMANTOWN	B	GLENDERSON	B	GOOSE CREEK	B	GRASSY BUTTE	A
GARCON	C	GERMANY	B	GLENDIVE	B	GOOSE CREEK, WET	C	GRASSYCONE	A

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GRAT	D	GRIVER, DRAINED	C	HAGEN	A	HANSEL	C	HATERTON	D
GRATTAN	A	GRIVER, CLAY	D	HAGENBARTH	B	HANSKA	C	HATHAWAY	B
GRAUFELS	C	SUBSTRATUM		HAGER	D	HANSKA,	B/D	HATLEY	C
GRAVDEN	D	GRIZZLY	B	HAGERMAN	C	DEPRESSIONAL		HATLIFF	C
GRAVELTON	B/D	GROBUTTE	B	HAGERSTOWN	C	HANSON	B	HATMAKER	C
GRAYBERT	B	GROGAN	B	HAGGA	B/D	HANTHO	B	HATPEAK	C
GRAYCALM	A	GROOM	C	HAGGA,	D	HANTZ	D	HATTIE	C
GRAYFORD	B	GROSECLOSE	C	SALINE-ALKALI		HANTZ, SALINE	D	HATTON	C
GRAYLAND	D	GROSS	C	HAGGERTY	B	HANTZ, DRY	C	HATUR	C
GRAYLAND, DRAINED	C	GROTON	A	HAGSTADT	B	HAP	B	HAUBSTADT	C
GRAYLING	A	GROTTO	A	HAGUE	A	HAPGOOD	B	HAUG	B/D
GRAYLOCK	A	GROUSEVILLE	C	HAIG	C/D	HAPJACK	D	HAUGAN	B
GRAYLOCK, STONY	B	GROVE	A	HAIGHTS	B	HAPNEY	C	HAULINGS	D
GRAYPOINT	B	GROVECITY	B	HAIKU	B	HARAHAN	D	HAUNCHEE	D
GRAYPOINT, WET	C	GROVER	B	HAILMAN	B	HARAHILL	C	HAUZ	C
GRAYROCK	C	GROVETON	B	HAIRE	C	HARANA	B	HAVALA	B
GRAYS	B	GROWDEN	B	HAKKER	C	HARBIN	B	HAVANA	B
GREAT BEND	B	GROWLER	B	HALACAN	D	HARBORD	B	HAVELOCK	B/D
GREEN BLUFF	B	GROWTON	B	HALAWA	B	HARCANY	B	HAVEN	B
GREEN CANYON	B	GRUBBS	D	HALBERT	D	HARCO	B	HAVERDAD	B
GREEN RIVER	C	GRUBSTAKE	B	HALDER	C	HARCOT	B/D	HAVERHILL	D
GREEN RIVER,	B	GRUENE	D	HALE	D	HARDEMAN	B	HAVERLY	C
STRONGLY SALINE		GRULLA	D	HALE, DRAINED	C	HARDESTY	B	HAVERSON	B
GREEN RIVER,	B	GRUMMIT	D	HALEDON	C	HARDING	D	HAVILLAH	B
FLOODED		GRUNDY	C	HALEIWA	B	HARDOL	B	HAVINGDON	C
GREEN RIVER, COOL	C	GRUVER	C	HALEY	B	HARDSCRABBLE	D	HAVRE	B
GREENBRAE	C	GRYGLA	B/D	HALF MOON	B	HARDTRIGGER	B	HAVRE, SALINE	C
GREENCREEK	B	GUADALUPE	B	HALFWAY	D	HARGILL	B	HAVRE, FLOODED,	B
GREENDALE	B	GUAJE	D	HALII	B	HARGREAVE	C	COOL	
GREENE	B	GUAMANI	B	HALIIMAILE	B	HARJO	D	HAVRE, FLOODED	B
GREENFIELD	B	GUANABANO	B	HALL	B	HARKERS	C	HAVRE, COOL	B
GREENFIELD,	C	GUANAJIBO	C	HALL RANCH	C	HARKEY	B	HAVRE, PE>31	B
HARDPAN		GUAYABOTA	D	HALLANDALE	B/D	HARKNESS	C	HAVRELON	B
SUBSTRATUM		GUAYAMA	D	HALLANDALE, TIDAL	D	HARLAN	B	HAW	B
GREENFIELD,	B	GUBE	C	HALLANDALE, SLOUGH	A/D	HARLEM	C	HAWI	B
GRAVELLY		GUBEN	B	HALLORAN	C	HARLESTON	C	HAWICK	A
GREENFIELD, COOL	B	GUCKEEN	C	HALSEY	C/D	HARLINGEN	D	HAWKEYE	A
GREENHALGH	B	GUDGREY	B	HAMACER	A	HARMEHL	C	HAWKINS	C
GREENHORN	D	GUELPH	B	HAMAKUAPOKO	B	HARMONY	C	HAWKSBILL	B
GREENLEAF	B	GUEMES	B	HAMAR	A/D	HARNEY	B	HAWKSPRINGS	B
GREENMAN	C	GUENOC	C	HAMBLEN	C	HAROL	D	HAWLEY	B
GREENOUGH	B	GUENTHER	B	HAMBONE	B	HARPER	D	HAWSLEY	A
GREENSON	C	GUERNSEY	C	HAMBRIGHT	D	HARPETH	B	HAXTUN	B
GREENTON	C	GUERRERO	A	HAMBURG	B	HARPS	B/D	HAYBOURNE	B
GREENVILLE	B	GUEST	D	HAMBY	C	HARPSTER	B/D	HAYCRIK	C
GREENVINE	D	GUFFIN	D	HAMDEN	B	HARPT	B	HAYDEN	B
GREENWATER	A	GUGUAK	D	HAMEL	C	HARQUA	C	HAYESTON	B
GREENWAY	B	GUILDER	C	HAMERLY	C	HARRIET	D	HAYESVILLE	B
GREENWOOD	A/D	GULER	B	HAMILTON	B	HARRIMAN	B	HAYESVILLE, STONY	C
GREHALEM	B	GULF	B/D	HAMLET	B	HARRINGTON	C	HAYFIELD	B
GRELL	D	GUMBLE	D	HAMLIN	B	HARRIS	D	HAYFORD	C
GRELLTON	B	GUMBOOT	D	HAMMACK	B	HARRISBURG	C	HAYHOOK	B
GRENADA	C	GUMBOOT, DRAINED	C	HAMMONTON	B	HARRISON	C	HAYMARKET	D
GRENADIER	B	GUNBARREL, SALINE	C	HAMPSHIRE	C	HARRISVILLE	C	HAYMOND	B
GRENVILLE	B	GUNBARREL, DRAINED	A	HAMPSON	C	HARROUN	D	HAYNESS	B
GRESHAM	C	GUND	C	HAMRE	C/D	HARSAN	B	HAYNIE	B
GRETDIVID	B	GUNDY	C	HAMTAH	C	HARSHA	B	HAYPRESS	A
GREWINGK	D	GUNN	B	HAMTAH, NONSTONY	B	HARSTINE	C	HAYSPUR	D
GREYBACK	B	GUNNEL	D	HAMTAH, COOL	B	HARSTON	B	HAYTER	B
GREYBO	B	GUNSIGHT	B	HANA	A	HART	D	HAYTI	D
GREYBULL	C	GUNSONE	D	HANAKER	C	HART CAMP	C	HAYWIRE	C
GREYEAGLE	D	GUNTER	B	HANALEI	C	HARTFORD	A	HAYWOOD	B
GREYS	B	GUP	C	HANAMAULU	B	HARTIG	B	HAZEL	C
GRIBBLE	C	GURDON	C	HANCEVILLE	B	HARTILL	C	HAZELAIR	D
GRIDELL	D	GURLEY	C	HAND	B	HARTLAND	B	HAZEN	B
GRIDGE	D	GUSTIN	C	HANDRAN	A	HARTLETON	B	HAZLEHURST	C
GRIDLEY	C	GUSTSPRING	B	HANDSBORO	D	HARTNIT	C	HAZLETON	B
GRIETA	B	GUTHRIE	D	HANDY	C	HARTSBURG	B/D	HAZTON	D
GRIEVES	B	GUY	B	HANDY, STONY	D	HARTSELLS	B	HEADQUARTERS	B
GRIFFITH	D	GUYTON	D	HANDY, NONFLOODED	C	HARTSHORN	B	HEAKE	D
GRIFFY	B	GWENA	D	HANEY	B	HARTVILLE	C	HEALDTON	D
GRIFTON	D	GWIN	D	HANFORD	B	HARTWELL	D	HEALING	B
GRIGSBY	B	GWINLY	D	HANGAARD	D	HARVARD	B	HEARNE	C
GRIGSTON	B	GWINNETT	B	HANGDO	B	HARVESTER	B	HEATH	C
GRIMM	A	GYMER	C	HANIPOE	B	HARVEY	B	HEATLY	A
GRIMM, STONY	B	GYNELLE	A	HANIS	C	HARVEY, BEDROCK	C	HEATON	A
GRIMSLEY	B	GYPNEVEE	B	HANKINS	C	SUBSTRATUM, DRY		HEBBRONVILLE	B
GRIMSTAD	B	GYSTRUM	C	HANKS	B	HARWOOD	C	HEBER	B
GRIMSTONE	B	HAAR	D	HANKSVILLE	D	HASKILL	A	HEBERT	C
GRINA	D	HACCKE	C	HANKSVILLE,	C	HASKINS	C	HEBO	D
GRINDBROOK	C	HACK	B	NONFLOODED		HASSEE	D	HEBRON	B
GRINDSTONE	C	HACKERS	B	HANLON	B	HASSELL	D	HECATE	B
GRINK	C	HACKROY	D	HANLY	A	HASTINGS	B	HECETA	B/D
GRISDALE	B	HACKWOOD	B	HANNA	B	HAT	C	HECHTMAN	D
GRISWOLD	B	HADAR	B	HANNAHATCHEE	B	HATBORO	D	HECKER	B
GRITNEY	C	HADES	B	HANNING	B	HATCH	D	HECLA	A
GRIVER	D	HADLEY	B	HANOVER	C	HATCHET	C	HECTOR	D
GRIVER, WET	D	HAFLINGER	A	HANS	C	HATERMUS	D	HEDOES	C

NOTES: TWO HYDROLOGIC SOIL GROUPS SUCH AS B/C INDICATES THE DRAINED/UNDRAINED SITUATION.
MODIFIERS SHOWN, E.G., BEDROCK SUBSTRATUM, REFER TO A SPECIFIC SOIL SERIES PHASE FOUND IN SOIL MAP LEGEND.

TABLE 7.1--HYDROLOGIC GROUPS OF THE SOILS OF THE UNITED STATES

HEDOX	C
HEDRICK	B
HEDVILLE	D
HEFED	B
HEFLIN	B
HEGLAR	B
HEGNE	D
HEIDEL	B
HEIDEN	D
HEIDTNAN	C
HEIGHTS	B/D
HEIL	D
HEINDAL	B
HEINSAW	C
HEISETON	B
HEISETON, STONY	C
HEISETON, SALINE-ALKALI	C
HEISETON, DRAINED	B
HEISETON, FLOODED	B
HEISLER	B
HEIST	B
HEITT	C
HEIZER	D
HELDT	C
HELEMANO	B
HELENA	C
HELENDALE	B
HELLMAN	C
HELMER	C
HELMICK	D
HELTER	B
HELVETIA	C
HELY	C
HEMBRE	B
HEMINGFORD	B
HENCO	B/D
HENDERSON	B
HENDRICKS	B
HENDY	C
HENEFER	C
HENHOLT	B
HENKIN	B
HENLEY	C
HENLINE	C
HENMEL	C
HENNEKE	D
HENNEPIN	B
HENNESSY	B
HENNINGSEN	C
HENRIETTA	B/D
HENRIEVILLE	B
HENRY	D
HENSHAW	C
HENSLEY	D
HEPLER	C
HEPPSIE	D
HERAKLE	D
HERBERT	B
HERBMAN	D
HERD	C
HEREFORD	B
HERKIMER	B
HERN	C
HERMERING	B
HERMISTON	B
HERMON	A
HERNANDEZ	B
HERNDON	B
HERO	B
HEROD	D
HERRICK	B
HERSH	B
HERSHAL	D
HERTY	D
HESCH	B
HESPER	C
HESPERIA	B
HESPERUS	B
HESSEL	B/D
HESSELBERG	D
HESSELTINE	B
HESSING	B
HESSLAN	C
HESSON	C
HETERWA	C
HETTINGER	C/D
HEXT	B
HEYDER	B
HEYDLAUFF	B
HEYTOU	B
HEZEL	B
HI VISTA	C
HIARC	C
HIBAR	C
HIBBING	C
HIBERNIA	C
HICKNAN	B
HICKORY	C
HICKS	B
HICOTA	B
HIDALGO	B
HIDATSA	B
HIDEAWAY	D
HIDEWOOD	C
HIERRO	B
HIGGINS	D
HIGGINSVILLE	C
HIGH GAP	C
HIGHAMS	D
HIGHBANK	C
HIGHCAMP	B
HIGHFIELD	B
HIGHMORE	B
HIGHPOINT	D
HIGHTOWER	C
HIGHWOOD	C
HIHIMANU	B
HIIBNER	C
HIKO PEAK	B
HIKO SPRINGS	B
HILAIRE	B
HILDEBRECHT	C
HILDRETH	D
HILEA	D
HILES	B
HILGER	B
HILGRAVE	B
HILLBRICK	D
HILLCO	B
HILLEMANN	C
HILLERY	D
HILLET	B/D
HILLFIELD	B
HILLGATE	D
HILLIARD	B
HILLIARD, MODERATELY WELL DRAINED	C
HILLON	C
HILLSBORO	B
HILLSDALE	B
HILLTO	B
HILLWOOD	B
HILMAR	D
HILMAR, DRAINED	B
HILMOE	C
HILO	A
HILOLO	D
HILT	B
HILTON	B
HINCKLEY	A
HINDES	C
HINESBURG	C
HINKER	C
HINKLE	D
HINMAN	C
HIRSCHDALE	C
HISEGA	C
HISER	B
HISLE	D
HITILO	A
HITT	B
HIVAL	D
HIWAN	D
HIWASSEE	B
HIWOOD	A
HIXTON	B
HOADLY	C
HOBACKER	B
HOBAN	B
HOBBS	B
HOBCAW	D
HOBE	A
HOBERG	C
HOBIT	C
HOBOG	D
HOBONNY	D
HOBSON	C
HOCAR	D
HOCHHEIM	B
HOCKINSON	D
HOCKINSON, MODERATELY WET	C
HOCKINSON, DRAINED	B
HOCKLEY	C
HOCKLEY, GRADED	D
HODA	C
HODEDO	C
HODENPYL	B
HODGE	A
HODGINS	B
HODGSON	C
HOFFLAND	D
HOFFMANVILLE	C
HOFFSTADT	B
HOGADERO	B
HOGANSBURG	B
HOGG	C
HOGNALAT	D
HOGRIS	B
HOH	B
HOHMANN	C
HOKAH	B
HOKO	B
HOLBROOK	B
HOLCOMB	D
HOLDAWAY	D
HOLDER	B
HOLDERMAN	C
HOLDERNESS	C
HOLDINGFORD	C
HOLDREGE	B
HOLILLIPAH	A
HOLLAND	B
HOLLANDLAKE	C
HOLLENBECK	D
HOLLINGER	B
HOLLIS	C/D
HOLLISTER	D
HOLLOMAN	C
HOLLOMEX	B
HOLLOWAY	B
HOLLY	B/D
HOLLY, PONDED	D
HOLLY SPRINGS	D
HOLLYWELL	B
HOLLYWOOD	D
HOLMAN	A
HOLMDEL	C
HOLMES	B
HOLOMUA	B
HOLOPAW	B/D
HOLSINE	B
HOLSTEIN	B
HOLSTON	B
HOLT	B
HOLTER	B
HOLTLE	B
HOLTON	C
HOLTVILLE	C
HOLYOKE	C/D
HONA	C
HOME CAMP	C
HOMELAKE	B
HOMELAND	C
HOMER	B
HOMESTAKE	C
HOMESTEAD	B
HOMME	C
HOMME, MODERATELY WET	B
HOMOSASSA	D
HONAUNAU	A
HONCUT	B
HONDALE	D
HONDOHO	B
HONEOYE	B
HONEYGROVE	C
HONEYVILLE	C
HONN	B
HONOBIA	C
HONOKAA	A
HONOLUA	B
HONOMANU	A
HONONEGAH	A
HONOULIULI	D
HONTAS	B
HONTOON	B/D
HONUAULU	A
HOOD	B
HOODLE	B
HOODOO	D
HOODSPORT	C
HOOGDAL	C
HOOKS	B
HOOLEHUA	B
HOOLY	C
HOOPAL	D
HOOPER	D
HOOPESTON	B
HOOSAN	B
HOOSIC	A
HOOSIERVILLE	C
HOOT	D
HOOTEN	D
HOPCO	C
HOPDRAW	A
HOPEKA	D
HOPKINS	B
HOPLAND	B
HOPLEY	B
HOPSONVILLE	C
HOQUIAM	B
HORD	B
HOREB	C
HOREB, GRAVELLY SUBSTRATUM	B
HORNELL	D
HORNING	B
HORNITOS	D
HORNSBY	C
HORNSVILLE	C
HORROCKS	B
HORSECAMP	D
HORSESHOE	B
HORSETHIEF	B
HORSLEY	D
HORTONVILLE	B
HOSKIN	C
HOSKINNINI	D
HOSLEY	D
HOSMER	C
HOSPAL	D
HOSSICK	B
HOSTAGE	B
HOT LAKE	C
HOTAW	C
HOTCREEK	D
HOTEL	B
HOTSPRINGS	B
HOUDEK	B
HOUGH	B
HOUGHTON	A/D
HOUGHTON, NAAT>50	A/D
HOUGHTON, PONDED	D
HOUGHTON, FREQUENTLY FLOODED	A/D
HOUGHTONVILLE	C
HOUK	C
HOULA	B
HOULKA	D
HOURGLASS	B
HOUSE MOUNTAIN	D
HOUSER	D
HOUSTAKE	C
HOUSTON	D
HOUSTON BLACK	D
HOVDE	C
HOVEN	D
HOVENWEEP	C
HOVERT	D
HOVEY	C
HOWARD	A
HOWCAN	B
HOWE	C
HOWELL	C
HOWLAND	C
HOWSON	C
HOYE	B
HOYLETON	C
HOYPUS	A
HOYTVILLE	C/D
HUALAPAI	C
HUB	B
HUBBARD	A
HUBBARDTON	C
HUBBELL	B
HUBERLY	D
HUBERT	B
HUBLERSBURG	B
HUCKLEBERRY	C
HUDSON	C
HUECO	C
HUEL	A
HUENEME	C
HUENEME, DRAINED	B
HUERFANO	D
HUEY	D
HUFFINE	B
HUFFMAN	B
HUFFTON	B
HUGGINS	C
HUGHES	B
HUGHESVILLE	C
HUGO	B
HUGUSTON	D
HUICHICA	C
HUICHICA, PONDED	D
HUIKAU	A
HUKILL	B
HULETT	B
HULLS	C
HULLT	B
HULUA	D
HUM	B
HUMACAO	B
HUMATAS	C
HUMBARGER	B
HUMBIG	C
HUMBIRD	B
HUMBOLDT	D
HUMBOLDT, MODERATELY WET, SALINE-ALKALI	B
HUMBOLDT, MODERATELY WET, SALINE	B
HUMBOLDT, SALINE	D
HUMBOLDT, MODERATELY WET	B
HUMBOLDT, DRAINED	B
HUNDUN	B
HUMESTON	C/D
HUMMINGTON	C
HUMPHREYS	B
HUMPTULIPS	B
HUNSKEL	C
HUN	B
HUNCHBACK	D
HUNEWILL	B
HUNGRY	C
HUNNTON	C
HUNTERS	B
HUNTINER	C
HUNTING	C
HUNTINGTON	B
HUNTMOUNT	B
HUNTSBURG	D
HUNTSVILLE	B
HUPP	B
HURDS	B
HURLBUT	C
HURLEY	D
HURRICANE	C
HURST	D
HUSE	D
HUSSA, CLAYEY SUBSTRATUM	D
HUSSA, SALINE	D
HUSSA, MODERATELY WET	C
HUSSA, DRAINED	B
HUSSA, SANDY SUBSTRATUM	D
HUSSMAN	D
HUSUM	B
HUTCHINSON	C
HUTSON	B

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TABLE 7.1--HYDROLOGIC GROUPS OF THE SOILS OF THE UNITED STATES

HUTT	D	INSKIP	C	JACOBSEN	O	JOBPEAK	D	KACHEMAK	B
HUTTON	C	INSULA	O	JACOBY	C	JOCITY	B	KACHESS	B
HUXLEY	D	INTERIOR	B	JACOT	B	JOCKO	B	KADE	C
HUYSINK	B	INTON	B	JACQUES	C	JODERO	B	KADLETZ	B
HYANNIS	B	INVERNESS	B	JACQUITH	C	JOEL	B	KADOKA	B
HYAS	B	INVERSHIEL	C	JACRATZ	O	JOENEY	D	KAENA	O
HYATTVILLE	C	INVILLE	B	JACWIN	B	JOES	B	KAFING	B
HYDE	B/D	IO	B	JADIS	B	JOHNNIE	C	KAHALUU	D
HYDER	D	IOLEAU	C	JAGUEYES	B	JDHNS	C	KAHANA	B
HYDRO	C	IONA	B	JAL	B	JDHNSBURG	D	KAHANUI	B
HYE	B	IONIA	B	JALNAR	A	JOHNSON	B	KAHLER	B
HYLOC	D	IOSCO	B	JANES	O	JOHNSTON	D	KAHLOTUS	B
HYNAS	D	IOSEPA	O	JANES CANYON	C	JOHNSWOOD	B	KAHOLA	B
HYRUN	B	IOTLA	B	JANES CANYON,	B	JOICE	D	KAHUA	D
HYSHAN	D	IPAGE	A	ORAINED		JOINER	B	KAIDERS	B
IAO	B	IPANO	C	JANESTON	C/O	JOKDDOWSKI	D	KAILUA	A
IBERIA	D	IPAVA	B	JANISE	C	JOLAN	D	KAIMU	A
ICARIA	D	IPISH	C	JANISE, OVERBLOWN,	B	JOLIET	D	KAINALIU	A
ICENE	O	IPSON	B	DRAINEO		JDNALE	B	KAIPOIOI	B
ICHBOD	D	IPSWICH	D	JANISE, DRAINED	C	JONAS	B	KAIWIKI	A
ICHETUCKNEE	D	IRA	C	JANISE, OVERBLOWN	C	JONATHAN	B	KALAE	B
ICICLE	B	IRAAN	B	JANSEN	B	JONCA	C	KALALOCH	B
IOA	B	IREDELL	C/D	JANUDE	B	JONDA	B	KALANA	C
IOABEL	B	IRELAND	C	JANUDE, CLAY	C	JONES	B	KALANAZOO	B
IOANONT	B	IRETEBA	B	SUBSTRATUN		JONESVILLE	B	KALAPA	B
IOEE	C	IRIGUL	D	JARAB	D	JOPLIN	C	KALAUPAPA	O
IOLEWILD	D	IRIN	C	JAREALES	D	JOPPA	B	KALIFONSKY	D
IOLEWILD, DRAINED	C	IRMULCD	B	JARITA	C	JORDAN	D	KALIGA	B/D
IONON	B	IROCK	C	JARNILLO	B	JORGE	B	KALIHI	D
IGDELL	C	IRON BLOSSON	C	JAROLA	D	JORY	C	KALISPELL	B
IGERT	C	IRON MOUNTAIN	D	JAROSD	C	JOSBURG	C	KALKASKA	A
IGNACIO	C	IRON RIVER	B	JARRE	B	JOSEPHINE	B	KALLIO	C
IGO	D	IRONCO	B	JARRON	D	JDSHUA	C	KALNARVILLE	B/D
IGUALDAO	D	IRONDALE	C	JASCO	O	JOSIE	B	KALNIA	B
IHLEN	B	IRONTDN	C	JASON	D	JOSLIN	B	KALOKO	D
IJAM	D	IROQUDIS	B/O	JASPER	B	JOURDANTON	B	KALONA	C
ILACHETONEL	D	IRRAWADDY	C	JAUCAS	A	JOWEC	D	KALSIN	D
ILOEFONSO	B	IRRIGDN	C	JAVA	B	JOY	B	KALSTED	B
ILES	C	IRVINE	O	JAWBONE	O	JUAB	B	KAMACK	B
ILIFF	C	IRVINGTON	C	JAY	C	JUANA DIAZ	B	KAMAKOA	B
ILIILI	D	IRWIN	D	JAYAR	C	JUBILEE	D	KANAN	D
ILION	D	ISAAC	C	JAYBEE	O	JUBILEE, CLAYEY	D	KANAOA	B
ILLABOT	B	ISABELLA	B	JAYEL	O	JUBILEE, WET	D	KAMAOLE	B
ILLER	B	ISAN	A/D	JAYEN	B	JUBILEE, DRAINED	B	KANAY	D
ILTON	C	ISANTI	A/O	JAYNES	D	JUBILEE, FLOODED	D	KANELA	C
ILWACO	B	ISBELL	B	JEAN	A	JUBILEE, GRAVELLY	D	KANIE	B
INA	B	ISHI PISHI	C	JEAN LAKE	B	JUDA	B	KANPVILLE	C
IMBLER	B	ISHPEMING	A	JEANERETTE	O	JUDD	C	KANRAR	B
IMLAY	D	ISKNAT	C	JEBO	B	JUDELL	B	KANAKA	B
IMNIG	C	ISLAND	B	JEDD	C	JUDICE	D	KANAPAHA	B/D
INNOKALEE	B/D	ISLOTE	B	JEDDO	C/D	JUDITH	B	KANARANZI	B
INNDKALEE,	D	ISOLDE	A	JEFFERS	B/O	JUDITH, BEDROCK	C/D	KANARRA	O
DEPRESSIONAL		ISON	B	JEFFERSON	B	SUBSTRATUN		KANAWHA	B
INNOKALEE,	B/O	ISTER	C	JEFFREY	B	JUDITH, GRAVELLY	B	KANDALY	A
LINESTONE		ISTOKPOGA	B/D	JEKLEY	C	JUDITH, COBBLY	B	KANDIK	B
SUBSTRATUN		ITANO	C	JEMEZ	C	JUOKINS	C	KANE	B
INOGENE	D	ITASCA	B	JENA	B	JUDSON	B	KANEBREAK	C
INONIL	B	ITAT	B	JENKINS	C	JUDY	C	KANEOHE	B
INPACT	A	ITCA	D	JENKINSON	O	JUG	A	KANEPUU	B
INPERIAL	D	ITHACA	C	JENNESS	B	JUGET	D	KANER	A
INAVALE	A	ITSWDOT	B	JENNINGS	C	JUGHANOLE	B	KANGAS	A
INCELL	D	IUKA	C	JENNY	O	JUGSON	C	KANID	B
INCHAU	C	IVA	C	JERAG	O	JULES	B	KANIKSU	B
INCY	A	IVAN	B	JERAULD	D	JULESBURG	A	KANIMA	C
INDART	C	IVANELL	C	JERICHO	D	JULIN	D	KANKAKEE	B
INDIAHOMA	D	IVER	B	JEROME	D	JUMBO	B	KANLEE	C
INDIAN CREEK	D	IVES	B	JERRY	C	JUNPE	B	KANONA	D
INDIANO	C	IVES, WET	O	JERRYSLU	C	JUNPER	C	KANOSH	C
INDIANOLA	A	IVES, FLOODED	B	JERU	B	JUMPOFF	C	KANTISHNA	D
INDIO	B	IVINS	C	JESSE CAMP	B	JUNCAL	C	KANUTCHAN	O
INDUS	D	IYRES	O	JESSUP	C	JUNCOS	D	KANZA	D
INEZ	D	IZAGORA	C	JETSTER	C	JUNCTION	B	KAPAA	B
INGALLS	B	IZEE	C	JETT	B	JUNEAU	B	KAPAPALA	B
INGENIO	B	IZO	A	JEWETT	B	JUNG	D	KAPAPALA, BEDROCK	C
INGERSOLL	B	IZUSER	B	JIGGS	B	JUNIPERBUTE	A	SUBSTRATUM	
INGRAN	O	JABU	B	JILSON	D	JUNIPERO	B	KAPIN	C
INKLER	B	JACAGUAS	B	JIM	C	JUNIUS	C	KAPOD	B
INKOM	D	JACANA	O	JINBO	B	JUNKETT	C	KAPOWSIN	D
INKOM, DRAINEO	C	JACEE	C	JIMEK	C	JUNO	A	KAPTURE	B
INKOSR	D	JACINTO	B	JIMENEZ	C	JUNQUITOS	C	KAPUHIKANI	D
INKS	D	JACK CREEK	A	JIMLAKE	B	JUPITER	B/D	KARANIN	B
INKSTER	B	JACKET	C	JINSAGE	B	JURA	D	KARANKAWA	O
INMACHUK	D	JACKNAN	B	JIMTOWN	C	JURVANNAH	C	KARCAL	O
INMAN	C	JACKNIFE	C	JIPPER	B	JUSTESEN	C	KAROE	B
INNO	A	JACKPORT	O	JIVAS	B	JUSTIN	B	KARLAN	C
ININGER	C	JACKS	C	JOACHEN	O	JUVA	B	KARLIN	A
INPENDENCE	B	JACKSON	B	JOB	C	JUVAN	D	KARLO	D
INSAK	D	JACOB	O	JOBOS	C	KAALUALU	A	KARLSBURG	B

NOTES: TWO HYOROLOGIC SOIL GROUPS SUCH AS B/C INOICATES THE ORAINEO/UNDRAINED SITUATION.
MOOIFIERS SHOWN, E.G., BEDROCK SUBSTRATUN, REFER TO A SPECIFIC SOIL SERIES PHASE FOUND IN SOIL NAP LEGENO.

TABLE 7.1--HYDROLOGIC GROUPS OF THE SOILS OF THE UNITED STATES

KARLSTAD	A	KENAI	C	KILLPACK	C	KITTSON	C	KONERT, DRAINED	C
KARLUK	D	KENANSVILLE	A	KILMANAGH	C	KIVA	A	KONNER	D
KARMA	B	KENDAIA	C	KILMER	C	KIWANIS	B	KONNER, DRAINED	C
KARNAK	D	KENDALL	B	KILMERQUE	C	KIZHUYAK	B	KONOCTI	B
KARNES	B	KENDALLVILLE	B	KILN	D	KJAR	D	KONSIL	B
KARRO	B	KENDRICK	A	KILOA	A	KLABER	D	KOOLAU	C
KARS	A	KENESAW	B	KILOHANA	A	KLABER, DRAINED	C	KOONICH	A
KARSHNER	D	KENMOOR	B	KILOWAN	C	KLADNICK	A	KOONTZ	D
KARTAR	B	KENN	B	KILWINNING	D	KLAMATH	D	KOOSHAREM	B
KASEBERG	D	KENNAN	B	KIM	B	KLAUS	B	KOOSKIA	C
KASHWITNA	B	KENNEBEC	B	KIM,	B	KLAWASI	D	KOOTENAI	B
KASILOF	B	KENNER	D	ELEVATION>6500		KLAWATTI	C	KOPIE	D
KASKI	B	KENNEWICK	B	KIM, SALINE	C	KLAWHOP	B	KOPPERL	B
KASOTA	C	KENNEY	A	KIM, COOL	B	KLAYENT	C	KOPPES	B
KASSLER	A	KENNEY LAKE	C	KIMAMA	B	KLEINBUSH	D	KORCHEA	B
KASSON	C	KENO	D	KIMBALL	D	KLEJ	B	KORENT	B
KATAMA	B	KENOMA	D	KIMBERLINA	B	KLICKER	C	KORNMAN	B
KATEMCY	C	KENSAL	B	KIMBERLY	B	KLICKITAT	B	KOROBAGO	C
KATHER	C	KENSETT	B	KIMBROUGH	D	KLICKSON	B	KORONIS	B
KATO	B/D	KENSPUR	B	KIMMERLING	D	KLINE, COBBLY	B	KORTTY	B
KATSEANES	D	KENT	D	KINO	C	KLINE, PROTECTED	A	KOSCIUSKO	B
KATULA	C	KENUSKY	D	KINA	D	KLINESVILLE	C/D	KOSETH	B
KATY	D	KENYON	B	KINCHELOE	D	KLINGER	B	KOSSE	B
KAUDER	C	KEO	B	KINCO	A	KLONE	B	KOSSUTH	B/D
KAUFMAN	D	KEOKUK	B	KINDER	C	KLOOCHMAN	C	KOSZTA	B
KAUKAUNA	C	KEONAH	C	KINDIG	B	KLOOTCH	C	KOTO	D
KAUPO	A	KEOTA	B	KINDY	C	KLOOTCHIE	B	KOTZMAN	B
KAUPPI	B	KEOWNS	B/D	KINESAVA	B	KLOTEN	D	KOVICH	D
KAVETT	D	KEPLER	C	KINGFISHER	B	KLUG	B	KOYEN	B
KAWAIHAE	C	KERBER	B	KINGHORN	D	KLUM	B	KOYNIK	D
KAWAIHAPAI	B	KERBY	B	KINGILE	C	KLUMP	B	KOYUKUK	B
KAWBAWGAM	C	KERMIT	A	KINGINGHAM	C	KLUTINA	B	KRADE	B
KAWICH	A	KERNAN	C	KINGMAN	D	KNAPKE	B	KRAKON	D
KAWKAWLIN	C	KERRICK	B	KINGMONT	B	KNAPPA	B	KRAM	D
KAYO	B	KERSHAW	A	KINGS	D	KNAPPTON	B	KRANSKI	B
KEAAU	D	KERSICK	D	KINGSBURY	D	KNEELAND	C	KRANZBURG	B
KEAHUA	B	KERSTON	A/D	KINGSDOWN	B	KNIFFIN	C	KRATKA	B/D
KEALAKEKUA	A	KERT	C	KINGSLAND	A/D	KNIGHT	B/D	KRAUSE	B
KEALIA	D	KESSLER	C	KINGSLEY	B	KNIK	B	KREAMER	C
KEARL	C	KESSON	D	KINGSTON	B	KNIKLIK	B	KREBS	B
KEARNS	B	KESTERSON	D	KINGSVILLE	A/D	KNIPPA	C	KREM	A
KEARSARGE	B	KESWICK	C	KINGTAIN	B	KNOB HILL	B	KREMLIN	B
KEATING	C	KETCHLY	B	KINKEAD	C	KNOBTOP	C	KRESSON	C
KEAUKAHA	D	KETCHUM	B	KINKEL	C	KNOCO	D	KREYENHAGEN	B
KEAWAKAPU	B	KETONA	D	KINKEL, GRAVELLY	B	KNOKE	B/D	KRIER	D
KEBLER	B	KETTENBACH	C	KINKORA	D	KNOLLE	B	KRIEST	B
KECH	D	KETTLE	B	KINMAN	C	KNOTT	D	KRONEN	B
KECKO	B	KETTLEMAN	B	KINNEAR	B	KNOWLES	B	KRUEGER	B
KECKSROAD	C	KETTNER	C	KINNEY	B	KNOX	B	KRUM	D
KEDA	B	KEUTERVILLE	B	KINROSS	A/D	KNULL	B	KRUSE	B
KEDRON	C	KEVANTON	B	KINSTON	B/D	KNUTSEN	B	KUBE	B
KEECHELUS	C	KEVIN	C	KINTA	D	KOBAR	C	KUBLER	C
KEECHI	C	KEWAUNEE	C	KINTON	C	KOBEH	B	KUBLI	D
KEEFERS	C	KEWEENAW	A	KINZEL	B	KOBEL	D	KUCERA	B
KEEI	D	KEYA	B	KIOMATIA	A	KOCH	D	KUCK	C
KEEKEE	B	KEYES	D	KIONA	B	KOCH, DRAINED	C	KUDLAC	D
KEEL	C	KEYNER	D	KIOWA	B	KODAK	B	KUHL	D
KEELDAR	B	KEYPORT	C	KIPER	B	KODIAK	B	KUKAIAU	A
KEELE	B	KEYSTONE	A	KIPLING	D	KOORA	C	KUKAIAU, BEDROCK	C
KEENE	C	KEZAN	D	KIPPEN	A	KOEHLER	C	SUBSTRATUM	
KEENO	C	KEZAR	B	KIPSON	D	KOELE	B	KULA	B
KEESE	D	KIAKUS	C	KIRBY	A	KOEPKE	B	KULLIT	B
KEESEMA	C	KIAMICHI	D	KIRBYVILLE	B	KOERLING	B	KULSHAN	C
KEEWATIN	C	KIAWAH	B/D	KIRK	D	KOERTH	C	KUMA	B
KEG	B	KIBBIE	B	KIRKENDALL	B	KOETHER	D	KUNATON	D
KEGEL	C	KIBESILLAH	B	KIRKHAM	C	KOFA	D	KUNAYOSH	A
KEGEL, DRAINED	D	KICKAPOO	B	KIRKLAND	D	KOGISH	D	KUNIA	B
KEGONSA	B	KICKERVILLE	B	KIRKSEY	C	KOHALA	B	KUNUWEIA	B
KEHENA	C	KIDD	D	KIRKVILLE	C	KOKAN	A	KUNZ	B
KEHOE	B	KIDDER	B	KIRLEY	C	KOKEE	B	KUPREANOF	A
KEIGLEY	B	KIDMAN	B	KIRTLEY	C	KOKERNOT	C	KUREB	A
KEISER	B	KIEHL	A	KIRVIN	C	KOKO	B	KURO	D
KEITH	B	KIESEL	C	KIRVIN, GRADED	D	KOKOKAHI	D	KURTZ	C
KEITHVILLE	D	KIETZKE	D	KISATCHIE	D	KOKOMO	B/D	KUSKOKWIM	D
KEKAHA	B	KIEV	B	KISHONA	B	KOLAR	D	KUSLINA	D
KEKAKE	D	KIKONI	B	KISHONA, ALKALI	C	KOLBERG	C	KUTCH	C
KELK	C	KILAGA	C	KISRING	C	KOLEKOLE	C	KUTLER	C
KELLER	C	KILARC	D	KISRING, WET	D	KOLIN	C	KUY	A
KELLY	D	KILAUEA	B	KISSICK	C	KOLLS	D	KVICHAK	B
KELSO	C	KILBURN	B	KISTIRN	B	KOLLUTUK	D	KWED	A
KELTNER	B	KILCHIS	C	KITCHELL	B	KOLOA	C	KYBURZ	B
KELTYS	B	KILDOR	C	KITCHEN CREEK	B	KOLOB	C	KYDAKA	D
KELVIN	C	KILFOIL	C	KITI	D	KOLOKOLO	B	KYLE	D
KEMAN	B	KILGORE	D	KITSAP	C	KOLOMOKI	B	KYLER	D
KEMMERER	C	KILKENNY	B	KITTERLL	D	KONO	B	LA BRIER	D
KEMOO	B	KILLBUCK	C/D	KITTITAS	D	KONA	D	LA FARGE	B
KEMP	C	KILLDUFF	B	KITTITAS, DRAINED	C	KONAWA	B	LA FONDA	B
KEMPSVILLE	B	KILLEY	D	KITTREDGE	B	KONERT	D	LA GRANDE	C

NOTES: TWO HYDROLOGIC SOIL GROUPS SUCH AS B/C INDICATES THE DRAINED/UNDRAINED SITUATION.
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TABLE 7.1--HYDROLOGIC GROUPS OF THE SOILS OF THE UNITED STATES

LA HOGUE	B	LANAWA	B	LARRY, DRAINED	C	LEATHAM	C	LETHENT	D
LA LANDE	B	LAMBERT	B	LARSON	D	LEATHERMAN	D	LETNEY	A
LA PALMA	C	LAMBETH	C	LARTON	A	LEAVENWORTH	B	LETON	D
LA POSTA	B	LAMBNAN	D	LARUE	A	LEAVITT	B	LETORT	B
LA PRAIRIE	B	LAMBRING	B	LARUSH	B	LEAVITTVILLE	B	LETRI	B/D
LA ROSE	B	LAMEDEER	B	LARVIE	D	LEBAM	B	LETTIA	B
LABENZO	B	LAMINGTON	D	LAS	C	LEBANON	C	LEVASY	C
LABETTE	C	LAMKIN	B	LAS ANIMAS	C	LEBEAU	D	LEVELTON	D
LABISH	D	LAMO	C	LAS FLORES	D	LEBEC	B	LEVERETT	C
LABISKI	B	LAMOILLE	B	LAS LUCAS	C	LEBO	B	LEVIATHAN	B
LABOU	D	LAMONDI	B	LAS POSAS	C	LEBSACK	C	LEVY	D
LABOUNTY	D	LAMONI	C	LAS VEGAS	D	LECK KILL	B	LEW	B
LABRE	B	LAMONT	B	LASA	A	LEDFORD	B	LEWIS	D
LABSHAFT	D	LAMONTA	D	LASALLE	D	LEDGEFORK	A	LEWISBERRY	B
LABU	D	LAMOTTE	B	LASAUSES	D	LEDMOUNT	D	LEWISBURG	C
LABUCK	B	LAMOURE	C	LASIL	D	LEDOW	A	LEWISTON	C
LACAMAS	D	LAMPHIER	B	LASSEL	C	LEDRU	D	LEWISVILLE	B
LACAMAS, DRAINED	C	LAMPSHIRE	D	LASSEN	D	LEDUB	B	LEWKALB	C
LACERDA	D	LAMSON	B/D	LASSITER	B	LEDUCK	C	LEX	B
LACHAPELLA	D	LANARK	B	LASTANCE	B	LEDWITH	B/D	LEXINGTON	B
LACITA	B	LANCASTER	B	LATAH	C	LEE	D	LEXTON	B
LACKAWANNA	C	LANCE	B	LATAHCO	C	LEEBENCH	D	LEYBA	B
LACLEDE	B	LAND	C	LATANIER	D	LEEDS	C	LEYDEN	C
LACONNER	C	LAND, WET	D	LATCH	A	LEEFIELD	C	LIBBINGS	D
LACOOCHEE	D	LAND, DRAINED	B	LATENE	B	LEELANAU	A	LIBEG	B
LACOSTE	C	LANDCO	C	LATES	C	LEEMONT	D	LIBERAL	D
LACOTA	B/D	LANDER	B	LATHAM	D	LEEPER	D	LIBORY	A
LACRESCENT	B	LANDES	B	LATHER	D	LEERAY	D	LIBRARY	D
LACY	D	LANDLOW	C	LATHROP	B	LEESBURG	B	LIBUSE	C
LADD	B	LANDMAN	B	LATINA	D	LEETONIA	C	LICK	B
LADELLE	B	LANE	C	LATIUM	D	LEEVAN	D	LICKDALE	D
LADNER	D	LANEY	B	LATON	D	LEFOR	B	LICKING	C
LADOGA	B	LANG, CLAYEY	C	LATONIA	B	LEGAULT	D	LICKSKILLET	D
LADUE	B	SUBSTRATUM		LATOUCHE	D	LEGLER	B	LICKSKILLET, STONY	D
LADYSMITH	D	LANG, MODERATELY	B	LATOURELL	B	LEGORE	B	LICKSKILLET,	C
LAFE	D	WET		LATTAS	D	LEHEW	C	NONSTONY	
LAFITTE	D	LANGFORD	C	LATTY	D	LEHIGH	C	LIDDELL	B/D
LAG	B	LANGHEI	B	LAUDERDALE	D	LEHMANS	D	LIDDIEVILLE	B
LAGLORIA	B	LANGLADE	B	LAUDERHILL	B/D	LEHR	B	LIDY	B
LAGNAF	B	LANGLOIS	D	LAUGENOUR	B	LEICESTER	C	LIEN	D
LAGONDA	C	LANGOLA	B	LAUGHLIN	C	LEIDL	C	LIGGET	B
LAGRANGE	D	LANGRELL	B	LAUNAIA	B	LEIGHCAN	B	LIGHTNING	D
LAGROSS	A	LANGSPRING	B	LAUREL	D	LEILEHUA	B	LIGNUM	C
LAGUNITA	A	LANGSTON	B	LAURELWOOD	B	LEISY	B	LIGON	D
LAGUNITA	A	LANGTRY	D	LAUREN	B	LELA	D	LIGURTA	B
LAGUNITA, STRONGLY	A	LANIER	A	LAURENTZEN	B	LELAND	D	LIHEN	A
SALINE		LANIGER	B	LAVACREEK	B	LEMAH	A	LIHUE	B
LAGUNITA, WET	C	LANKBUSH	B	LAVALLEE	B	LEMCO	C	LIKES	A
LAHAINA	B	LANKIN	C	LAVATE	B	LEMERT	D	LILAH	A
LAHONTAN	D	LANKTREE	C	LAVEEN	B	LEMETA	D	LILBERT	B
LAHRITY	C	LANOAK	B	LAVERKIN	C	LEMING	C	LILBOURN	B
LAIDIG	C	LANSDALE	B	LAVIC	B	LEMM	B	LILLINGTON	B
LAIDLAW	B	LANSDOWNE	C	LAVINA	D	LEMOND	B/D	LILY	B
LAIL	C	LANSING	B	LAWAI	B	LEMONEX	C	LIMA	B
LAIRO	B	LANTERN	B	LAWET	B/D	LEMOORE	C	LIMBER	B
LAIRDSVILLE	D	LANTIS	B	LAWLER	B	LEN	C	LIMERICK	C
LAJARA	D	LANTON	D	LAWNDALE	B	LENA	A/D	LIMON	C
LAJITAS	D	LANTONIA	B	LAWNWOOD	B/D	LENAPAH	D	LIMON, WET	D
LAKE	A	LANTRY	B	LAWRENCE	C	LENAWEE	B/D	LIMON, NONFLOODED	C
LAKE CHARLES	D	LANTZ	D	LAWRENCEVILLE	C	LENAWEE, PONDED	D	LIMONES	B
LAKE CREEK	C	LANVER	C	LAWSHE	D	LENBERG	C	LIMPIA	C
LAKE JANEE	B	LANYON	C/D	LAWSON	C	LENNEP	C	LINCO	B
LAKEHELEN	C	LAP	D	LAWTHER	D	LENOIR	D	LINCOLN	A
LAKEHURST	A	LAPARITA	C	LAWTON	C	LENZ	B	LINDAAS	C/D
LAKELAND	A	LAPDUN	B	LAWYER	B	LENZ, STONY	C	LINDALE	C
LAKEMONT	D	LAPED	D	LAX	C	LENZBURG	B	LINDELL	C
LAKEPORT	B	LAPEER	B	LAXAL	B	LEO	B	LINDEN	B
LAKESHORE	D	LAPHAM	A	LAXTON	C	LEOLA	B	LINDER	B
LAKESIDE	C	LAPINE	A	LAYCOCK	B	LEON	B/D	LINDLEY	C
LAKESOL	B	LAPLATTA	C	LAYTON	A	LEON, OCCASIONALLY	A/D	LINDRITH	B
LAKETON	B	LAPON	D	LAYVIEW	D	FLOODED		LINDSIDE	C
LAKEVIEW	C	LAPORTE	D	LAZEAR	D	LEONARD	D	LINDSTROM	B
LAKEWIN	B	LAPOSA	C	LE BAR	B	LEONARDO	B	LINDY	C
LAKEWOOD	A	LARAND	B	LE SUEUR	B	LEONARDTOWN	D	LINE	B
LAKI	B	LARCHMOUNT	B	LEA	C	LEONI	B	LINEVILLE	C
LAKIN	A	LARDELL	C	LEADER	B	LERDAL	C	LINGANORE	B
LAKOA	B	LAREDO	B	LEADPOINT	B	LERDO	C	LINHART	A
LAKOMA	D	LARES	C	LEADVALE	C	LEROY	B	LININGER	C
LAKRIDGE	C	LARGO	B	LEADVILLE	B	LERROW	C	LINKER	B
LALAAU	A	LARIAT	B	LEAF	D	LESHARA	B	LINKVILLE	B
LALINDA	B	LARIM	B	LEAFU	C	LESHO	C	LINNE	C
LALLIE	D	LARIMER	B	LEAGUEVILLE	B/D	LESLIE	D	LINNET	C
LAN	D	LARKIN	B	LEAKSVILLE	D	LESON	D	LINNEUS	B
LANA	C	LARKSON	C	LEAL	B	LESTER	B	LINO	B
LAMANGA	C	LARMINE	D	LEALANDIC	D	LESWILL	B	LINOYER	B
LAMAR	B	LAROQUE	B	LEANNA	D	LETA	C	LINROSE	C
LAMARSH	C	LAROSE	D	LEANTO	D	LETCHER	D	LINSLAW	D
LAMARTINE	C	LARRY	D	LEAPS	C	LETHA	C	LINT	B

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LINTON	B	LOLEKAA	B	LOUSCOT	B	LYFORD	C	MAKAPILI	B
LINVELDT	B	LOLO	B	LOUVIERS	D	LYKENS	C	MAKAWAO	B
LINVILLE	B	LOLON	B	LOVEJOY	C	LYLES	B/D	MAKAWELI	B
LINWELL	C	LOMA	C	LOVELACE	B	LYMAN	C/D	MAKENA	B
LINWOOD	A/D	LOMAKI	B	LOVELAND	C	LYMANSON	C	MAKI	C
LIPAN	D	LOMALTA	D	LOVELL	D	LYME	C	MAKIKI	B
LIPKE	D	LOMART	B	LOVELOCK	B/D	LYNCH	D	MAKLAK	A
LIPPINCOTT	B/D	LOMAX	B	LOVELOCK, SALINE	B	LYNCHBURG	C	MAKOTI	B
LIPPITT	C	LOMETA	C	LOVELOCK,	B	LYNDEN	A	MAL	C
LIRIOS	B	LOMIRA	B	SALINE-ALKALI		LYNN HAVEN	B/D	MALA	B
LISADE	B	LOMITAS	D	LOVELOCK,	B	LYNNE	B/D	MALABAR	B/D
LISAM	D	LOMOINE	D	MODERATELY WET		LYNNVILLE	C	MALABON	C
LISBON	B	LOMOND	B	LOVEWELL	B	LYNNWOOD	A	MALACHY	B
LISCO	C	LOMPICO	B	LOVLINE	C	LYNX	B	MALAGA	A
LISCOMB	B	LONCAN	C	LOWELL	C	LYONMAN	C	MALAMA	A
LISK	B	LONDO	C	LOWERCREEK	A	LYONS	D	MALARGO	B
LISMAS	D	LONDONDERRY	C/D	LOWLEIN	B	LYONSVILLE	B	MALAYA	D
LISMORE	B	LONE	C	LOWNDES	B	LYRA	D	MALBIS	B
LITCHFIELD	A	LONE ROCK	A	LOWRY	B	LYRE	B	MALCOLM	B
LITHGOW	C	LONEPINE	B	LOWS	B/D	LYSTAIR	B	MALDEN	A
LITIMBER	B	LONERIDGE	C	LOWVILLE	B	LYTELL	B	MALEZA	B
LITLE	D	LONERIDGE, COOL	B	LOX	C	LYVILLE	B	MALHEUR	C
LITRO	D	LONESTAR	B	LOXLEY	A/D	MABANK	D	MALIBU	D
LITTLE HORN	C	LONETREE	A	LOYAL	B	MABEN	C	MALIN	D
LITTLE POLE	D	LONEWOOD	B	LOYALTON	D	MABI	D	MALJAMAR	B
LITTLE WOOD	B	LONGCREEK	D	LOYSVILLE	D	MABRAY	D	MALM	C
LITTLEBEAR	B	LONGDOE		LOZA	D	MACADD	D	MALO	B
LITTLEJOHN	C	LONGFORD	C	LOZANO	B	MACAR	B	MALOTERRE	D
LITTLETON	B	LONGJIM	D	LOZIER	D	MACAREEND	C	MALOY	B
LITTSAN	C	LONGLOIS	B	LUALUALEI	D	MACE	B	MALPAIS	B
LITZ	C	LONGMARE	D	LUANA	B	MACEDONIA	B	MALSTROM	B
LIV	D	LONGMONT	C	LUBBOCK	B	MACFARLANE	B	MALVERN	C
LIVENGOOD	B	LONGRIE	B	LUBRECHT	C	MACHETE	C	MAMALA	D
LIVERMORE	B	LONGVAL	B	LUCAS	D	MACHIAS	B	MAMOU	C
LIVIA	D	LONGVIEW	C	LUCEDALE	B	MACHUELO	D	MANAHAA	C
LIVINGSTON	D	LONIGAN	B	LUCERNE	B	MACK	C	MANAHAWKIN	D
LIVONA	B	LONJON	B	LUCERO	B	MACKEN	D	MANAMA	C
LIZZANT	B	LONNA	B	LUCIEN	C	MACKEY	C	MANARD	C
LLANOS	C	LONOKE	B	LUCILE	A	MACKINAC	B	MANASSA	C
LOARC	B	LONTI	D	LUCKENBACH	C	MACKSBURG	B	MANASSAS	B
LOBDELL	B	LOOKINGGLASS	C	LUCKIAMUTE	D	MACNEAL	B	MANATEE	D
LOBELVILLE	C	LOOKOUT	C	LUCKY	C	MACOMB	B	MANAWA	C
LOBERG	C	LOOMER	D	LUCKY STAR	B	MACOMBER	C	MANBURN	D
LOBERT	B	LOOMIS	D	LUCKYRICH	B	MACON	B	MANCELONA	A
LOBITOS	C	LOPER	C	LUCY	A	MADALIN	D	MANCHESTER	A
LOBO	D	LOPEZ	D	LUDDEN	D	MADAWASKA	B	MANDAN	B
LOBURN	D	LOPWASH	B	LUDINGTON	B	MADDEN	C	MANDARIN	B/D
LOCANE	D	LORACK	B	LUDLOW	C	MADDOCK	A	MANDERFIELD	B
LOCEY	C	LORADALE	C	LUEDERS	C	MADELIA	B/D	MANDEVILLE	B
LOCHLOOSA	C	LORAIN	C/D	LUFKIN	D	MADELINE	D	MANET	B
LOCHSA	B	LORAN	B	LUGERT	B	MADERA	D	MANFRED	D
LOCKE	B	LOROSTOWN	C	LUHON	B	MADGE	B	MANGUM	D
LOCKERBY	C	LOREAUVILLE	C	LUKE	C	MADILL	B	MANHATTAN	A
LOCKHART	B	LORELLA	D	LULA	B	MADISON	B	MANHEIM	C
LOCKMAN	B	LORENA	B	LULING	D	MADONNA	C	MANI	C
LOCKPORT	D	LORENZO	B	LULUDE	B	MADRAK	C	MANILA	C
LOCKTON	B	LORETTO	B	LUMBEE	B/D	MADRAS	C	MANISTEE	A
LOCKWOOD	B	LORING	C	LUMMI	D	MADRID	B	MANITA	C
LOCKWOOD, WET	C	LORMAN	D	LUMMI, DRAINED	C	MADRONE	C	MANITOWISH	B
LOCO	C	LOS ALAMOS	B	LUNA	C	MADUREZ	B	MANLEY	B
LOCODA	D	LOS BANOS	C	LUNCH	C	MAES	C	MANLIUS	C
LOCUST	C	LOS GATOS	C	LUNDER	D	MAGALLON	A	MANN	B/D
LODALLEY	D	LOS GATOS,	B	LUNDS	C	MAGENS	B	MANNING	B
LODAR	D	GRAVELLY		LUNDY	D	MAGGIE	D	MANOGUE	D
LODI	B	LOS GUINEOS	C	LUNT	C	MAGGIN	C	MANOR	B
LODO	D	LOS OSOS	C	LUPE	B	MAGHILLS	B	MANSELO	B
LOFFTUS	C	LOS ROBLES	B	LUPINTO	B	MAGIC	D	MANSFIELD	D
LOFTON	D	LOS TANOS	C	LUPINTO, SALINE	C	MAGINNIS	D	MANSIC	B
LOGAN	D	LOSEE	B	LUPOYOMA	B	MAGNA	D	MANSKER	B
LOGDELL	B	LOSTINE	B	LUPPINO	D	MAGNOR	C	MANTACHIE	C
LOGGERT	A	LOSTVALLY	C	LUPTON	A/D	MAGNUS	C	MANTECA	C
LOGHOUSE	B	LOSTWELLS	B	LURA	C/D	MAGOTSU	D	MANTEO	C/D
LOGY	B	LOSTWELLS, WET	D	LURAY	C/D	MAHALASVILLE	B/D	MANTER	B
LOHLER	C	LOTHAIR	C	LURNICK	C	MAHANA	B	MANU	C
LOHMILLER	C	LOTT	C	LUTE	D	MAHASKA	B	MANVEL	B
LOHNES	A	LOUDERBACK	C	LUTH	C	MAHONING	D	MANVEL, SALINE	C
LOHSMAN	D	LOUDON	C	LUTHER	B	MAHTOMEDI	A	MANZANITA	C
LOIRE	B	LOUDONVILLE	C	LUTIE	B	MAHTOWA	C/D	MANZANO	B
LOKEN	B	LOUELLA	B	LUTON	D	MAHUKONA	B	MANZANOLA	C
LOKERN	C	LOUGHBORO	C	LUTTERLOH	C	MAIA	B	MAPLE MOUNTAIN	B
LOKERN,	D	LOUGHTY	B	LUTZKE	B	MAIDEN	C	MAPLETON	C/D
SALINE-ALKALI,		LOUIE	C	LUVERNE	C	MAILE	A	MARAGLADE	C
WET		LOUIN	D	LUXOR	D	MAINSTAY	D	MARAGUEZ	B
LOKERN,	D	LOUISA	B	LUZENA	D	MAITLAND	B	MARATHON	B
SALINE-ALKALI		LOUISBURG	B	LYBROOK	D	MAJADA	B	MARBLE	A
LOKOSEE	B/D	LOUP	D	LYDA	D	MAKAALAE	B	MARBLEMOUNT	B
LOLAK	D	LOUPLOUP	B	LYDICK	B	MAKAH	B	MARCELINAS	D
LOLALITA	B	LOURDES	C	LYERLY	D	MAKALAPA	D	MARCELLON	C

NOTES: TWO HYDROLOGIC SOIL GROUPS SUCH AS B/C INDICATES THE DRAINED/UNDRAINED SITUATION.
MODIFIERS SHOWN, E.G., BEDROCK SUBSTRATUM, REFER TO A SPECIFIC SOIL SERIES PHASE FOUND IN SOIL MAP LEGEND.

TABLE 7.1--HYDROLOGIC GROUPS OF THE SOILS OF THE UNITED STATES

MARCETTA	B	MARVYN	B	MAYODAN	B	MCLAIN	C	MENOON	B
MARCIAL	D	MARY	C	MAYOWORTH	C	MCLAURIN	B	MENDOTA	B
MARCOLA	C	MARYSLAND	B/D	MAYQUEEN	A	MCLEOD	B	MENEFEE	D
MARCONI	C	MASADA	C	MAYSDORF	B	MCLOUGHLIN	B	MENFRO	B
MARCOTT	C	MASARDIS	A	MAYTOWN	B	MCMEEN	C	MENLO	D
MARCUM	C	MASARYK	A	MAYVILLE	B	MCMULLIN	C	MENO	C
MARCUS	B/D	MASCAMP	D	MAYWOOD	B	MCMULLIN, WARM	D	MENOKEN	C
MARCUS, ALKALI,	D	MASCARENAS	C	MAZASKA	C/D	MCMURDIE	C	MENOMINEE	A
WET		MASCHETAH	B	MAZUMA	B	MCMURRAY	D	MENTO	C
MARCUSE	D	MASCOTTE	B/D	MC BETH	D	MCMURRAY, DRAINED	C	MENTOR	B
MARCY	D	MASCOTTE,	B	MC CORT	B	MCNARY	D	MENZEL	B
MARDIN	C	DEPRESSIONAL		MCAFEE	C	MCNEAL	B	MEQUON	C
MARENGO	C/D	MASCOTTE,	B/D	MCALLEN	C	MCNULL	C	MER ROUGE	B
MARESUA	B	OCCASIONALLY		MCALLISTER	C	MCNULTY	B	MERCED	D
MARGATE	B/D	FLOODED		MCALPIN	C	MCPAUL	B	MERCEDES	D
MARGERUM	B	MASET	B	MCBEE	C	MCPHIE	B	MERCER	C
MARGO	B	MASHAM	D	MCBEE, LOAMY	B	MCQUARRIE	D	MERCEY	C
MARIA, DRAINED	B	MASHEL	C	SUBSTRATUM		MCQUEEN	C	MERDEN	D
MARIA, FLOODED	B	MASHULAVILLE	B/D	MCBIGGAM	C	MCRAE	B	MEREDITH	B
MARIA, CLAY	C	MASKELL	B	MCBRIDE	B	MCRAVEN	C	MERETA	C
SUBSTRATUM		MASON	B	MCCAFFERY	A	MCVEGAS	D	MERGEL	B
MARIANA	C	MASONFORT	D	MCCAIN	C	MCVICKERS	C	MERIDIAN	B
MARIAS	D	MASSANETTA	B	MCCALEB	B	MEAD	D	MERINO	D
MARIAVILLE	D	MASSBACH	B	MCCALL	B	MEADIN	A	MERKEL	B
MARICAO	B	MASSENA	C	MCCALLY	D	MEADLAND	C	MERLIN	D
MARICOPA	B	MASSIE	D	MCCAMMON	C	MEADOWCREEK	C	MERMILL	B/D
MARIETTA	C	MASTERSON	B	MCCANN	B	MEADOWLAKE	C	MERNA	B
MARILLA	C	MATAGORDA	D	MCCAREY	C	MEADOWVILLE	B	MEROS	A
MARIMEL	C	MATAMOROS	C	MCCARRAN	B	MECAN	B	MERRICK	B
MARIMEL, DRAINED	B	MATANUSKA	B	MCCARTHY	B	MECHANICSBURG	C	MERRILL	C
MARINA	B	MATANZAS	B	MCCASH	B	MECKESVILLE	C	MERRILLAN	C
MARINE	C	MATAPEAKE	B	MCCLAVE	C	MECKLENBURG	C	MERRIMAC	A
MARION	D	MATAWAN	C	MCCLEARY	D	MECOSTA	A	MERRITT	B
MARIPOSA	C	MATCHER	A	MCCLELLAN	B	MEDA	B	MERSHON	B
MARISCAL	D	MATFIELD	C	MCCLOUD	C	MEDANO	C	MERTON	B
MARISSA	C	MATHERS	B	MCCLURE	C	MEDANO, FLOODED	D	MERTZ	C
MARKES	D	MATHERTON	B	MCCOIN	D	MEDARY	C	MERWIN	A/D
MARKESAN	B	MATHESON	B	MCCOLL	D	MEDBURN	B	MESA	B
MARKET	D	MATHIAS	B	MCCOLLUM	B	MEDCO	D	MESABA	C
MARKEY	A/D	MATHIS	C	MCCONNEL	B	MEDFORD	B	MESCAL	C
MARKHAM	C	MATHISTON	C	MCCOOK	B	MEDFRA	D	MESCALERO	C
MARKLAND	C	MATHON	B	MCCORT	B	MEDICINE	B	MESEI	D
MARKTON	C	MATOY	C	MCCOY	C	MEDLEY	B	MESPUN	A
MARLA	D	MATTAMUSKEET	D	MCCOYSBURG	B	MEDLIN	D	MESSER	C
MARLAKE	D	MATTAPEX	C	MCCREE	B	MEDOMAK	D	MET	B
MARLBORO	B	MATTAPONI	C	MCCRORY	D	MEDORA	B	METAMORA	B
MARLEAN	B	MATUNUCK	D	MCCROSKET	B	MEDWAY	B	METCALF	D
MARLETTE	B	MAU	C	MCCULLOUGH	B	MEEGERO	B	METEA	B
MARLOW	C	MAUDE	B	MCCULLY	C	MEEHAN	B	METH	C
MARLTON	C	MAUGHAN	C	MCCUMBER	B	MEEKS	B	METIGOSHE	B
MARMARTH	B	MAUKEY	C	MCCUNE	D	MEETEETSE	D	METOLIUS	B
MARNA	D	MAULDIN	D	MCDANIEL	B	MEGALOS	D	METRE	D
MAROSA	B	MAUMEE	A/D	MCDOLE	B	MEGGETT	D	METZ	A
MARPA	C	MAUNABO	D	MCDONALD	C	MEGONOT	C	METZ, SILTY	B
MARQUETTE	A	MAUPIN	C	MCDONALDSVILLE	C/D	MEGUIN	B	SUBSTRATUM	
MARQUEZ	C	MAUREPAS	D	MCDUFF	C	MEHLHORN	C	METZ, FLOODED	A
MARR	B	MAURICE	B	MCELROY	B	MEIKLE	D	METZ, GRAVELLY	B
MARRIOTT	B	MAURY	B	MCEWEN	B	MEISS	D	SUBSTRATUM	
MARSDEN	C	MAUSER	B	MCFADDEN	B	MELAND	C	MEXICO	D
MARSEILLES	B	MAUVAIS	C	MCFAIN	C	MELBOURNE	B	MEXISPRING	D
MARSELL	B	MAVCO	C	MCFARLAND	B	MELBY	C	MEYSTRE	B
MARSHALL	B	MAVERICK	C	MCFAUL	C	MELD	C	MHOON	D
MARSHAN	B/D	MAVIE	B/D	MCGAFFEY	B	MELDER	B	MIAMI	B
MARSHDALE	C	MAWAE	A	MCGARR	C	MELGA	D	MIAMIAN	C
MARSHDALE	D	MAWER	B	MCGARY	C	MELHOMES	D	MICANOPY	C
MARSHDALE, DRAINED	C	MAX	B	MCGEHEE	C	MELITA	A	MICCO	B/D
MARSHDALE, COOL	D	MAXCREEK	B/D	MCGILVERY	A	MELLENTHIN	D	MICHELSON	B
MARSHFIELD	B/D	MAXEY	C	MCGINNIS	B	MELLOR	D	MICHIGAMME	B
MARSING	B	MAXFIELD	B/D	MCGINTY	B	MELLOR, WET	D	MICHIGAMME,	C
MART	B	MAXTON	B	MCGIRK	C	MELLOR, DRY	C	MODERATELY WET	
MARTEL	D	MAXVILLE	B	MCGIRK, LOW	D	MELLOTT	B	MICHIGAMME, COBBLY	C
MARTELLA	B	MAXWELL	D	PRECIPITATION		MELOCHE	D	MICKEY	D
MARTIN	C	MAY	B	MCGOWAN	B	MELOLAND	C	MIDAS	B
MARTIN PENA	D	MAY DAY	D	MCGRATH	B	MELON	B	MIDCO	A
MARTINECK	D	MAYACAMA	C	MCGREW	B	MELROSE	C	MIDDLE	C
MARTINEZ	D	MAYBERRY	D	MCGUIRE	B	MELTON	D	MIDDLEBURY	B
MARTINI	B	MAYBID	D	MCHENRY	B	MELVILLE	B	MIDDLETOWN	B
MARTINSBURG	B	MAYDOL	B	MCILWAINE	B	MELVIN	D	MIDDLEWOOD	D
MARTINSDALE	B	MAYER	B/D	MCINTOSH	B	MEMALOOSE	C	MIDELIGHT	C
MARTINSON	C	MAYES	D	MCINTYRE	B	MEMPHIS	B	MIDESSA	B
MARTINSVILLE	B	MAYFIELD	B	MCKAMIE	D	MENAHGA	A	MIDFORK	B
MARTINTON	C	MAYFLOWER	C	MCKAY	D	MENARD	B	MIDLAND	D
MARTIS	B	MAYGER	C	MCKENNA	D	MENASHA	D	MIDMONT	C
MARTISCO	B/D	MAYHEW	D	MCKENNA, DRAINED	C	MENBO	C	MIDNIGHT	D
MARTY	B	MAYMEAD	B	MCKENZIE	D	MENDEBOURE	C	MIDO	A
MARVAN	D	MAYMEN	D	MCKINLEY	B	MENDENHALL	D	MIDRAW	D
MARVELL	B	MAYNARD LAKE	A	MCKINNEY	C	MENDI	B	MIDVALE	C
MARVIN	C	MAYO	B	MCKNIGHT	B	MENDOCINO	B	MIDWAY	D

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TABLE 7.1--HYDROLOGIC GROUPS OF THE SOILS OF THE UNITED STATES

MIERHILL	C
MIESEN	D
MIESEN, DRAINED	C
MIFFLIN	B
MIGUEL	D
MIKE	D
MIKESELL	C
MIKIM	B
MIKKALO	C
MILAN	B
MILBURY	C
MILCAN	C
MILES	B
MILFORD	B/D
MILHAM	B
MILITARY	B
MILL HOLLOW	B
MILLADORE	C
MILLBORO	D
MILLBROOK	B
MILLBURNE	B
MILLER	D
MILLERLAKE	B
MILLERLUX	D
MILLERTON	D
MILLERVILLE	A/D
MILLETT	B
MILLGROVE	B/D
MILLHEIM	C
MILLHOPPER	A
MILLICH	D
MILLICOMA	C
MILLIGAN	C
MILLING	D
MILLINGTON	B/D
MILLIS	C
MILLPAW	C
MILLPOT	B
MILLROCK	A
MILLSAP	D
MILLSDALE	B/D
MILLSHOLM	D
MILLSITE	B
MILLVILLE	B
MILLWOOD	D
MILNER	B
MILPITAS	D
MILREN	C
MILTON	C
MIMBRES	B
MIMOSA	C
MINALOOSA	B
MINAT	B
MINATARE	D
MINCHEY	B
MINCO	B
MINDEN	B
MINE	B
MINER	D
MINERAL	C
MINERAL MOUNTAIN	C
MINGO	C
MINGUS	D
MINIDOKA	C
MINKLER	C
MINLITH	D
MINNEHA	C
MINNEISKA	B
MINNEOPA	B
MINNEOSA	B
MINNEQUA	B
MINNETONKA	D
MINNEWAUKAN	A/D
MINNIECE	D
MINNIEPEAK	A
MINNIMAUD	C
MINNITH	C
MINNYE	B
MINOA	C
MINOCQUA	B/D
MINTER	D
MINTO	C
MINU	D
MINVALE	B
MINVENO	D
MINWELLS	C
MION	D
MIPPON	A
MIRABAL	C
MIRACLE	B
MIRAGE	C
MIRAND	D
MIRANDA	D
MIRES	B
MIRKWOOD	D
MIRROR	B
MIRROR LAKE	A
MISAD	B
MISENHEIMER	B
MISHAK	C
MISSION	D
MISSISQUOI	A
MISSLER	B
MISSOULA	D
MITCH	B
MITCHELL	B
MITIWANGA	C
MITRE	C
MITTEN	B
MIVIDA	B
MIZEL	D
MOAG	D
MOANO	D
MOAPA	C
MOAULA	A
MOBATE	D
MOBEETIE	B
MOBERG	B
MOBRIDGE	B
MOCA	D
MOCAREY	D
MOCHO	B
MOCKLER	B
MOCMONT	B
MOCTILEME	D
MODA	D
MODALE	C
MODENA	B
MODESTO	C
MODJESKA	B
MODKIN	C
MODOC	C
MODYON	C
MOE	C
MOEN	C
MOENKOPIE	D
MOEPITZ	B
MOFFAT	B
MOGG	D
MOGLIA	C
MOGOLLON	B
MOGOTE	C
MOHALL	B
MOHAVE	B
MOHAWK	B
MOINGONA	B
MOKELUMNE	D
MOKENA	C
MOKIAK	B
MOKINS	D
MOKO	D
MOKULEIA	B
MOLALLA	B
MOLAND	B
MOLAS	D
MOLCAL	B
MOLENA	A
MOLION	D
MOLLICY	C
MOLLMAN	B
MOLLVILLE	D
MOLLY	B
MOLOKAI	B
MOLSON	B
MOLYNEUX	B
MOMOLI	B
MONA	B
MONACAN	C
MONACHE	B
MONAD	B
MONADNOCK	B
MONAHANS	B
MONARDA	D
MONASTERIO	B
MONAVILLE	B
MONDAMIN	C
MONDEY	C
MONDOVI	B
MONEE	D
MONICO	C
MONIDA	C
MONIERCO	D
MONITEAU	C/D
MONITOR	C
MONJEAU	D
MONOGRAM	B
MONONA	B
MONONGAHELA	C
MONROE	B
MONROEVILLE	C/D
MONSERATE	C
MONSERATE, THIN SURFACE	D
MONSON	C/D
MONTAGUE	D
MONTALTO	C
MONTARA	D
MONTAUK	C
MONTBORNE	B
MONTCALM	A
MONTE	B
MONTECITO	B
MONTEGRANDE	D
MONTELL	D
MONTELLO	C
MONTEOCHA	D
MONTEOLA	D
MONTEROSA	D
MONTESA	C
MONTEVALLO	D
MONTEZ	B
MONTGOMERY	D
MONTICELLO	B
MONTIETH	B
MONTLID	C
MONTMORENCI	B
MONTOSO	B
MONTOUR	D
MONTOYA	D
MONTOYA, OVERWASH	C
MONTOYA, FLOODED	D
MONTPELLIER	C
MONTROSS	C
MONTVALE	D
MONTVERDE	B/D
MONTWEL	B
MONUE	B
MOODY	B
MOOHOO	B
MOOLACK	A
MOONLIGHT	B
MOONSHINE	D
MOONSTONE	C
MOONVILLE	B
MOOREVILLE	C
MOOSE RIVER	D
MOOSELAKE	A/D
MOOSILAUKE	C
MOPANG	B
MOQUAH	B
MORA	C
MORADO	C
MORAN	B
MORD	C
MOREAU	D
MOREHEAD	C
MOREHOUSE	D
MORELAND	D
MORENO	C
MORET	D
MOREY	D
MORGALA	C
MORGANFIELD	B
MORIARTY	D
MORICAL	C
MORLEY	C
MORLING	D
MORMON MESA	D
MOROCCO	B
MORONI	D
MOROP	C
MORPH	B/D
MORRILL	B
MORRIS	C
MORRISON	B
MORRISTOWN	C
MORROW	C
MORSE	D
MORSET	B
MORTENSON	C
MORTON	B
MORVAL	B
MOSBY	C
MOSCA	B
MOSCOW	C
MOSEL	C
MOSHANNON	B
MOSHEIM	D
MOSHER	D
MOSHERVILLE	C
MOSIDA	B
MOSINEE	B
MOSLANDER	D
MOSO	B
MOSOMO	A
MOSQUET	D
MOSSYROCK	B
MOTA	B
MOTEN	C
MOTLEY	B
MOTOQUA	D
MOTT	B
MOTTLAND	B
MOTTO	D
MOTTSVILLE	A
MOULTON	C
MOULTRIE	D
MOUND	C
MOUNDPRAIRIE	B/D
MOUNDPRAIRIE, PONDED	D
MOUNDVILLE	A
MOUNT HOME	B
MOUNT LUCAS	C
MOUNTAINBOY	D
MOUNTAINBURG	D
MOUNTAINEER	C
MOUNTAINVIEW	C
MOUNTAINVILLE	B
MOUNTVIEW	B
MOVILLE	C
MOWATA	D
MOWEBA	B
MOWER	C
MOWICH	C
MOXEE	D
MOYERS	C
MOYERSON	D
MOYINA	D
MROW	C
MT. AIRY	A
MT. CARROLL	B
MT. VERNON	C
MUCARA	D
MUCKALEE	D
MUD SPRINGS	C
MUDRAY	D
MUDSOCK	B/D
MUES	C
MUFF	D
MUGGINS	C
MUGHOUSE	C
MUGHUT	C
MUIR	B
MUIRKIRK	B
MUKILTEO	D
MUKILTEO, PONDED	D
MUKILTEO, DRAINED	C
MULAT	D
MULDOON	B
MULDROW	D
MULETT	D
MULGON	B
MULKEY	C
MULLICA	C
MULLIG	B
MULLINS	D
MULLYON	D
MULSHOE	C
MULSTAY	C
MULT	C
MULTNOMAH	B
MULTORPOR	A
MUNDAL	C
MUNDELEIN	B
MUNDEN	B
MUNDOS	B
MUNDT	C
MUNI	D
MUNISING	B
MUNJOR	B
MUNK	C
MUNSET	D
MUNSON	D
MUNUSCONG	B/D
MURAD	C
MURDO	B
MURDOCK	C
MUREN	B
MURNEN	B
MUROC	D
MURPHY	C
MURRIETA	D
MURRILL	B
MURVILLE	A/D
MUSCATINE	B
MUSE	C
MUSELLA	B
MUSICK	B
MUSINIA	B
MUSKEGO	A/D
MUSKEGO	D
MUSKEGO, OVERWASH	A/D
MUSKINGUM	C
MUSKOGEE	C
MUSQUIZ	C
MUSSEL	B
MUSSELSHELL	B
MUSSEY	B/D
MUSTANG	A/D
MUTNALA	B
MUZZLER	D
MYAKKA	B/D
MYAKKA, DEPRESSIONAL	D
MYAKKA, SHELL SUBSTRATUM	B/D
MYAKKA, TIDAL	D
MYATT	D
MYERS	D
MYERSVILLE	B
MYFORD	D
MYLREA	C
MYOMA	A
MYOMA, WET	B
MYRICK	C
MYRTLE	B
MYSTEN	A
MYSTIC	C
NAALEHU	B
NAALEHU, BEDROCK SUBSTRATUM	C
NABESNA	D
NACHES	B
NACHUSA	B
NACIMIENTO	C
NACLINA	D
NACOGDOCHES	B
NADA	D
NADEAU	B
NADINA	D
NADRA	D
NAEGELIN	D
NAFF	B
NAGITSY	C
NAGLE	B
NAGROM	C
NAHATCHE	C
NAHMA	B/D
NAHON	D
NAHRUB	D
NAHUNTA	C
NAIWA	B
NAKAI	B
NAKARNA	B
NAKNEK	D
NALAKI	C
NALDO	B
NALL	D
NAMBE	B

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TABLE 7.1--HYDROLOGIC GROUPS OF THE SOILS OF THE UNITED STATES

NAMEOKI	D	NEENAH	C	NEWCOMB	A	NIWOT	C	NOVACAN	D
NANON	C	NEER	B	NEWDALE	B	NIXA	C	NOVARK	B
NAMUR	D	NEESOPAH	B	NEWELL	B	NIXON	B	NOVARY	B/D
NANAMKIN	A	NEETO	B	NEWELLTON	D	NIXONTON	B	NOVATO	D
NANCY	B	NEFF	C	NEWFLAT	D	NIZINA	A	NOVINA	B
NANIAK	D	NEGLEY	B	NEWFORK	D	NOARK	B	NOWATA	B
NANKIN	C	NEHALEN	B	NEWFOUND	C	NOBE	D	NOWEN	B/D
NANNY	B	NEHAR	B	NEWGLARUS	B	NOBLE	B	NOYER	B
NANNYTON	B	NEHAR, STONY	C	NEWHAN	A	NOBLETON	C	NOYES	C/D
NANSEMOND	C	NEIBER	C	NEWHOUSE	B	NOBSCOT	A	NOYO	C
NANSENE	B	NEICE	B	NEWKIRK	D	NOCKEN	C	NOYSON	C
NANSEPSEP	C	NEILTON	A	NEWLANDS	C	NODAWAY	B	NUBY	D
NANTUCKET	C	NEKIA	C	NEWLIN	B	NODEN	B	NUBY, DRAINED	C
NAPA	D	NEKKEN	B	NEWNAN	C	NODINE	B	NUC	C
NAPIER	B	NEKOMA	B	NEWNATA	C	NOELKE	D	NUCKOLLS	B
NAPLENE	B	NELDORE	D	NEWPASS	C	NOGAL	D	NUCLA	B
NAPOLEON	A/D	NELLA	B	NEWPORT	C	NOHILI	D	NUECES	C
NAPPANEE	D	NELLIS	B	NEWRY	B	NOKASIPPI	B/D	NUFF	C
NAPTOWNE	B	NELMAN	C	NEWSKAH	B	NOKAY	C	NUGENT	A
NARANJITO	C	NELSCOTT	B	NEWSON	A/D	NOKHU	C	NUKRUM	D
NARANJO	C	NELSON	B	NEWSROCK	B	NOLAM	B	NULEY	B
NARCISSE	B	NEMAOJI	B	NEWSTEAD	C	NOLICHUCKY	B	NULLIGAN	B
NARCOOSSEE	C	NENAH	D	NEWTON	A/D	NOLIN	B	NUMA	B
NARD	B	NENAH, DRAINED	C	NEWTONIA	B	NOLO	D	NUNDA	C
NAREL	B	NENICO	D	NEWTOWN	C	NOLTEN	D	NUNEMAKER	D
NARGAR	B	NENOTE	A	NEWULN	B	NOMARA	C	NUNICA	C
NARK	C	NENOURS	C	NEWVILLE	D	NOME	D	NUNN	C
NARLON	D	NENANA	B	NEYGAT	D	NONDALTON	B	NURKEY	B
NARON	B	NENNO	C	NEZ PERCE	C	NONOPAHU	D	NUSS	D
NARRAGANSETT	B	NEOTOMA	B	NGARDMAU	B	NONPAREIL	D	NUTIVOLI	A
NARROWS	D	NEPALTO	A	NGARDOK	B	NOOK	C	NUTLEY	C
NARTA	D	NEPESTA	B	NGATPANG	C	NOOKACHANPS	D	NUTRAS	C
NARU	C	NEPHI	C	NGEDEBUS	A	NOOKACHAMPS,	C	NUTRIOSO	B
NASER	B	NEPONSET	C	NGERSUUL	C	DRAINED		NUVALDE	B
NASH	B	NEPPEL	B	NGERUNGOR	D	NOOKSACK	C	NYALA	B
NASHOBA	C	NEPTUNE	A	NIAGARA	C	NOONAN	D	NYCON	A
NASHVILLE	B	NERESON	B	NIARADA	B	NORA	B	NYE	B
NASHWAUK	C	NESBITT	B	NIART	B	NORAD	B	NYJACK	C
NASON	C	NESOA	B	NIBLEY	C	NORBERT	D	NYMORE	A
NASS	D	NESHAMINY	B	NIBSON	D	NORBORNE	B	NYSSA	C
NASSAU	C	NESHOBA	C	NICANOR	D	NORCAN	C	NYSSATON	B
NASSET	B	NESIKA	B	NICASIO	D	NORD	B	O'BRIEN	B
NATAL	D	NESKAHI	B	NICHOLFLAT	D	NORDBY	B	O'LEARY	B
NATCHEZ	B	NESKOWIN	C	NICHOLIA	D	NORDEN	B	O'LENO	D
NATCHITOCHES	D	NESPELEN	B	NICHOLS	B	NORDIC	B	O'NEILL	B
NATHROP	B	NESPELEM, ALKALI	C	NICHOLSON	C	NORDICOL	B	OAHE	B
NATI	B	NESS	D	NICHOLVILLE	C	NORDNESS	B	OAK GLEN	B
NATIONAL	B	NESSEL	B	NICKEL	B	NORFOLK	B	OAK GROVE	C
NATKIN	B	NESTER	C	NICKIN	B	NORFORK	D	OAKALLA	B
NATROY	D	NESTUCCA	C	NICODEMUS	B	NORGE	B	OAKBORO	C
NATURITA	B	NET	C	NICODENUS, FLOODED	C	NORGO	D	OAKDEN	D
NAUMBURG	C	NETARTS	B	NICOLLET	B	NORKA	B	OAKES	B
NAUVOO	B	NETCONG	B	NIDO	C	NORLAND	B	OAKLAND	C
NAVACA	D	NETO	B	NIELSEN	D	NORMA	D	OAKLET	C
NAVACITY	B	NETRAC	A	NIGHTHAWK	B	NORMA, DRAINED	B	OAKLIMETER	C
NAVAJO	D	NETTLES	D	NIHILL	B	NORMA, GRAVELLY	D	OAKVILLE	A
NAVAN	D	NETTLETON	C	NIKEY	B	SUBSTRATUM		OAKWOOD	B
NAVO	D	NEUBERT	B	NIKFUL	D	NORMANGEE	D	OANAPUKA	B
NAWNEY	D	NEUNS	C	NIKISHKA	A	NORMANIA	B	OATMAN	B
NAWT	D	NEURALIA	C	NIKLASON	B	NOROB	C	OATUU	D
NAXING	B	NEUSKE	B	NIKOLAI	D	NORREST	C	OBAN	C
NAYE	C	NEVADOR	B	NILAND	C	NORRIE	B	OBANION	C
NAYPED	C	NEVARC	C	NILER	D	NORRIS	D	OBARO	B
NAYRIB	D	NEVAT	B	NINBRO	B	NORRISTON	A	OBEN	C
NAZ	B	NEVEE	B	NIMND	D	NORTE	C	OBISPO	D
NAZATON	B	NEVILLE	B	NINROD	C	NORTH POWDER	C	OBRAST	D
NEABSCO	C	NEVILLE, WET	C	NINUE	B	NORTHBORO	C	OBRAY	D
NEBAGO	C	NEVIN	B	NINEKAR	D	NORTHCASTLE	B	OBURN	D
NEBEKER	C	NEVINE	B	NINENILE	D	NORTHCOTE	C/D	OCALA	C
NEBGEN	D	NEVKA	C	NINEVEH	B	NORTHOALE	C	OCAMBEE	C
NEBISH	B	NEVOYER	D	NINIGRET	B	NORTHFIELD	D	OCANA	B
NEBONA	D	NEVTAH	C	NIOBELL	C	NORTHNORE	C	OCCOQUAN	B
NECANICUM	B	NEW CAMBRIA	C	NIOTA	D	NORTHWATER	B	OCCUM	B
NECESSITY	C	NEWALBIN	B/D	NIOTAZE	C	NORTON	C	OCEANET	D
NECHE	C	NEWALBIN, MUCK	D	NIPE	B	NORTONVILLE	C	OCEANO	A
NECTAR	C	SUBSTRATUN		NIPINTUCK	D	NORWAY	C	OCHEYEDAN	B
NEOA	C	NEWALBIN, PONDED	D	NIPPT	B	NORWELL	C	OCHLOCKONEE	B
NEDERLAND	B	NEWANNA	C	NIPSUN	C	NORWICH	D	OCHO	D
NEEDLE	D	NEWARK	C	NIRA	B	NORWOOD	B	OCHOCO	C
NEEDLE PEAK	C	NEWARK, PONDED	D	NISENE	B	NOSRAC	B	OCIE	C
NEEDLE PEAK,	B	NEWAUKUM	B	NISHNA	C/D	NOTAL	D	OCILLA	C
OCCASIONALLY		NEWAYGO	B	NISHON	D	NOTCHER	B	OCKLEY	B
FLOODED		NEWBELL	B	NISQUALLY	A	NOTI	D	OCOEE	B/D
NEEDLEYE	C	NEWBERG	B	NISULA	B	NOTTAWA	B	OCONEE	C
NEEDMORE	C	NEWBERN	C	NITTAW	D	NOTTER	B	OCONTO	B
NEELEY	B	NEWBERRY	C	NIU	B	NOTUS	C	OCOSTA	D
NEEN	C	NEWBON	B	NIULII	C	NOTUS, DRAINED	B	OCOSTA, DRAINED	C
NEEN, DRAINED	B	NEWCO	D	NIWANA	B	NOUQUE	D	OCQUEOC	A

NOTES: TWO HYDROLOGIC SOIL GROUPS SUCH AS B/C INDICATES THE DRAINED/UNDRAINED SITUATION.
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TABLE 7.1--HYDROLOGIC GROUPS OF THE SOILS OF THE UNITED STATES

OCQUEOC,	B	OLOMPALI	D	OROVADA	B	OXLEY	C	PALOS VERDES	D
MODERATELY WET		OLOT	B	ORPARK	C	OXWALL	D	PALOUSE	B
OCRAIG	D	OLOTANIA	A	ORPHANT	D	OYHUT	B	PALSGROVE	B
OCTAGON	B	OLPE	C	ORR	B	OZAMIS	D	PALUXY	B
OCTAVIA	B	OLSON	D	ORR, GRAVELLY	C	OZAN	D	PAMLICO	D
ODAS	D	OLTON	C	SUBSTRATUM		OZAUKEE	C	PAMOA	B
ODELL	B	OLUSTEE	B/D	ORRVILLE	C	OZETTE	C	PAMSDEL	C
ODEM	A	OLUSTEE, THICK	B	ORSA	A	PAAIKI	B	PAMUNKEY	B
ODENSON	D	SURFACE		ORSA, GRAVELLY	B	PAALOA	B	PANA	B
ODERMOTT	C	OLYIC	B	ORSET	B	PAAUHAU	A	PANAEWA	D
ODERMOTT, STONY	B	OLYMPIC	C	ORSINO	A	PABLO	D	PANAMA	B
ODESSA	D	OMADI	B	ORTEGA	A	PACHAPPA	B	PANAMINT	B
ODIN	C	OMEGA	A	ORTELLO	B	PACHECO	C	PANASOFFKEE	C/D
ODNE	D	OMENA	B	ORTING	C	PACHECO, DRAINED	B	PANCHERI	B
ODO	B	OMIO	B	ORTIZ	C	PACIFICO	C	PANDO	B
OELOP	B	OMNI	D	ORTON	B	PACK	C	PANDOAH	C
OEST	B	OMRO	C	ORWET	A/D	PACKARD	B	PANDORA	B/D
OESTERLE	C	ONSTOTT	C	ORWIG	B	PACKER	B	PANDURA	D
OFU	B	OMULGA	C	ORWOOD	B	PACKHAM	B	PANE	B
OGARTY	C	ONA	B/D	OSAGE	D	PACKTRAIL	C	PANGBORN	D
OGEECHEE	B/D	ONAMIA	B	OSAKIS	B	PACKWOOD	D	PANGUITCH	B
OGEMAW	C/D	ONAQUI	D	OSBORN	C	PACO	C	PANIN	B
OGILVIE	B/D	ONARGA	B	OSCAR	D	PACOLET	B	PANIOGUE	B
OGLALA	B	ONASON	D	OSCURA	C	PACTOLA	B	PANIOGUE, WET	C
OGLE	B	ONAWA	D	OSGOOD	A	PACTOLUS	A/C	PANITCHEN	B
OGRAL	B	ONAWAY	B	OSHA	B	PADDOCK	C/D	PANKY	C
OHACO	C	ONDAWA	B	OSHAWA	C/D	PADEN	C	PANMOO	C
OHANA	C	ONECO	B	OSHKOSH	C	PADILLA	C	PANOCHE	B
OHIA	A	ONEIL	C	OSHONE	C	PADINA	B	PANOCHE,	C
OIDEM	A	ONITA	C	OSHTEMO	B	PADUCAH	B	SALINE-ALKALI,	
OJATA	D	ONITE	B	OSIER	A/D	PADUS	B	WET	
OJIBWAY	C	ONOTA	B	OSITO	C	PAESL	B	PANOCHE,	B
OJITO	C	ONSLOW	B	OSKA	C	PAGEBROOK	D	SALINE-ALKALI	
OKANOGAN	B	ONTARIO	B	OSMUND	B	PAGET	C	PANOLA	D
OKATON	D	ONTKO	D	OSO	C	PAGODA	C	PANSEY	D
OKAW	D	ONTONAGON	D	OSOBB	D	PAGOSA	C	PANTANO	D
OKAY	B	ONYX	B	OSORIDGE	D	PAHOKEE	B/D	PANTEGO	B/D
OKEE	B	OOKALA	A	OSOTE	D	PAHRANAGAT	C	PANTERA	B
OKEECHOBEE	B/D	OOSEN	A	OSSIAN	B/D	PAHRANAGAT,	C	PANTHER	D
OKEELANTA	B/D	OPAL	D	OSSIPEE	D	DRAINED, SALINE		PANTON	D
OKEELANTA,	D	OPELIKA	D	OST	B	PAHRANAGAT, SALINE	C	PAOLA	A
DEPRESSIONAL		OPEQUON	C	OSTLER	C	PAHRANAGAT,	B	PAOLI	B
OKEELANTA, TIDAL	D	OPHIR	C	OSTRANDER	B	DRAINED		PAPAA	D
OKEETEE	D	OPIHIKAO	D	OSWALD	D	PAHRANGE	C	PAPAC	C
OKEMAH	C	OPLIN	C	OTEEN	C	PAHREAH	D	PAPAGUA	C
OKIOTA	D	OPPIO	D	OTERO	B	PAHROC	D	PAPAI	A
OKLARED	B	OQUAGA	C	OTHELLO	C/D	PAHRUMP	C	PAPALOTE	C
OKLAWAHA	B/D	ORA	C	OTISCO	A	PAHSIMEROI	B	PAPINEAU	C
OKO	D	ORACLE	D	OTISVILLE	A	PAIA	B	PARA	B
OKOBOJI	B/D	ORAID	A	OTLEY	B	PAICE	D	PARACHUTE	B
OKOLONA	D	ORAN	B	OTOMO	D	PAINESVILLE	C	PARADISE	C
OKREEK	D	ORANGE	D	OTOOLE	C	PAINT	D	PARANAT	C
OKTIBBEHA	D	ORANGEBURG	B	OTTER	B/D	PAISLEY	D	PARASOL	B
OLA	C	ORCAP	C	OTTERHOLT	B	PAIT	B	PARCELAS	D
OLAA	A	ORCAS	D	OTTERSON	A	PAJARA	C	PARCHIN	D
OLAC	D	ORCAS, DRAINED	C	OTTOKEE	A	PAJARITO	B	PARDALOE	B
OLAND	B	ORCHARD	B	OTTOSEN	B	PAJUELA	B	PARDEE	D
OLANTA	B	ORCKY	B	OTWAY	D	PAKA	B	PARDEEVILLE	B
OLASHES	B	ORD	B	OTWELL	C	PAKALA	B	PAREHAT	C
OLBUT	D	ORDNANCE	D	OUACHITA	C	PAKINI	B	PARENT	B/D
OLD CAMP	D	ORDWAY	D	OUARD	D	PALAFOX	C	PARISIAN	D
OLDHAM	C/D	OREJAS	D	OUPICO	C	PALANUSH	C	PARKAY	B
OLDS	D	ORELIA	D	OURAY	B	PALAPALAI	B	PARKDALE	B
OLDSMAR	B/D	ORELLA	D	OUSLEY	C	PALATINE	B	PARKE	B
OLELO	B	ORFORD	C	OUTLET	C	PALAU	B	PARKER	B
OLENTANGY	A/D	ORICTO	B	OUTLOOK	D	PALAZZO	C	PARKFIELD	C
OLEQUA	B	ORIDIA	D	OUTLOOK, DRAINED	C	PALINOR	C	PARKHILL	B/D
OLETE	C	ORIDIA, DRAINED	C	OVAN	D	PALISADE	B	PARKINSON	B
OLEX	B	ORIF	A	OVERGAARD	C	PALIX	B	PARKVIEW	B
OLGA	C	ORIGO	B	OVERLAND	C	PALLS	C	PARKVILLE	C
OLI	B	ORINOCO	C	OVERLY	C	PALM BEACH	A	PARKWOOD	B/D
OLIAGA	C	ORIO	B/D	OVERTON	D	PALMA	B	PARLEYS	B
OLIN	B	ORION	C	OVIATT	B	PALMAREJO	C	PARLIN	C
OLINDA	B	ORITA	B	OVID	C	PALMAS ALTAS	D	PARLO	B
OLIPHANT	B	ORIZABA	D	OVINA	B	PALMER CANYON	B	PARMELE	C
OLIVENHAIN	D	ORIZABA, WET	D	OWEGO	D	PALMERDALE	B	PARMELOW	C
OLIVIER	C	ORIZABA, DRAINED	C	OWEN CREEK	C	PALMETTO	B/D	PARMENTER	B
OLJETO	A	ORLA	B	OWENS	D	PALMETTO,	D	PARNELL	C/D
OLLEI	D	ORLAND	B	OWHI	B	DEPRESSIONAL		PARQUAT	B
OLMITO	D	ORLANDO	A	OWINZA	D	PALMICH	B	PARR	B
OLMITZ	B	ORLEANS	D	OWLCAN	B	PALMS	A/D	PARRAN	D
OLMOS	C	ORLIE	C	OWOSSO	B	PALMYRA	B	PARRISH	C
OLMSTED	B/D	ORMAS	B	OWSEL	B	PALODURO	B	PARRITA	D
OLNES	B	ORMSBY	C	OWYHEE	B	PALOMARIN	B	PARSHALL	B
OLNEY	B	ORNBAUN	C	OXBOW	C	PALOMAS	B	PARSIPPANY	C/D
OLOAVA	B	ORO FINO	B	OXCOREL	D	PALOMINO	D	PARSONS	D
OLOKUI	D	ORO GRANDE	D	OXERINE	C	PALON	B	PARTLOW	D
OLOMOUNT	C	ORONOCO	B	OXFORD	D	PALOPINTO	D	PARTOV	D

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PARTRI	C	PEGLEG	C	PERRY	D	PILLIKEN	B	PLACITAS	C
PASAGSHAK	D	PEGLER	D	PERRYPARK	B	PILLOT	B	PLACK	D
PASCO	D	PEGRAM	B	PERRYVILLE	B	PILLSBURY	C	PLAINBO	A
PASCO, DRAINED	C	PEKAY	C	PERSANTI	C	PILOT PEAK	D	PLAINFIELD	A
PASO SECO	D	PEKIN	C	PERSAYO	D	PILOT ROCK	C	PLAISTED	C
PASQUETTI	D	PELAN	B	PERSHING	C	PILTDOWN	B	PLANK	D
PASQUETTI,	C/D	PELEE	B	PERSIS	B	PILTZ	C	PLANKINTON	D
MODERATELY WET		PELELIU	D	PERT	D	PIMA	B	PLANO	B
PASQUETTI, DRAINED	C	PELHAM	B/D	PERU	C	PIMER	B	PLANTATION	B/D
PASQUOTANK	B/D	PELIC	D	PERVINA	C	PINAL	D	PLASKETT	D
PASS CANYON	D	PELION	B/D	PERWICK	C	PINALENO	B	PLATA	B
PASSAR	C	PELKIE	A	PESCADERO	D	PINAMT	B	PLATEA	C
PASSCREEK	C	PELLA	B/D	PESCAR	C	PINATA	C	PLATNER	C
PASTIK	B	PELLEJAS	B	PESERO	D	PINAVETES	A	PLATO	C
PASTORIUS	B	PELLICER	D	PESHASTIN	B	PINBIT	B	PLATORO	B
PASTURA	D	PELONCILLO	D	PESHEKEE	D	PINCHER	C	PLATTE	B
PATCHIN	D	PELTIER	C	PESO	C	PINCHOT	B	PLATTE, WET	D
PATE	C	PEMBERTON	B	PETACA	D	PINCKNEY	C	PLATTE, CHANNELED	D
PATENT	C	PEMBROKE	B	PETAL	C	PINCONNING	B/D	PLATTVILLE	B
PATHEAD	C	PEMENE	B	PETAN	D	PINE FLAT	B	PLAYCO	B
PATILLAS	B	PENA	B	PETEETNEET	D	PINEAL	D	PLAYER	D
PATILO	B	PENAPON	B	PETERMAN	D	PINEBUTTE	B	PLAYMOOR	C/D
PATIO	C	PENASCO	D	PETERS	D	PINEDA	B/D	PLAZA	C
PATIT CREEK	B	PENCE	B	PETERSON	B	PINEDA,	D	PLEASANT	C
PATMOS	C	PEND OREILLE	B	PETRIE	D	DEPRESSIONAL		PLEASANT, PONDED	D
PATNA	B	PENDARVIS	C	PETROLIA	B/D	PINEDA, LIMESTONE	B/D	PLEASANT, FLOODED	C
PATOS	C	PENDEN	B	PETROS	D	SUBSTRATUM		PLEASANT GROVE	B
PATOUTVILLE	C	PENDER	C	PETSPRING	D	PINEDALE	B	PLEASANT VALE	B
PATRICIA	B	PENDERGRASS	D	PETTIGREW	B/D	PINEGUEST	B	PLEASANT VIEW	B
PATRICK	B	PENDROY	D	PETTUS	C	PINEHURST	B	PLEASANTON	B
PATROLE	C	PENGILLY	B/D	PETTY	B	PINELLAS	B/D	PLEDGER	D
PATTANI	D	PENGRA	D	PEWAMO	C/D	PINELLI	B	PLEINE	D
PATTEE	B	PENINSULA	C	PEYTON	B	PINETOP	C	PLEIOVILLE	C
PATTENBURG	B	PENISTAJA	B	PFEIFFER	B	PINETUCKY	B	PLEITO	C
PATTER	B	PENITENTE	B	PHAGE	B	PINEVAL	B	PLEVNA	D
PATTERSON	C	PENLAW	C	PHALANX	B	PINEZ	B	PLITE	B
PATTON	B/D	PENN	C	PHANTOM	C	PINGREE	D	PLOME	B
PAUL	B	PENNEKAMP	A	PHARO	B	PINHOOK	B/D	PLOVER	C
PAULDING	D	PENNELL	D	PHARR	B	PINICON	B	PLUCK	C
PAULINA	D	PENNICHUCK	B	PHEBA	C	PINKEL	C	PLUMMER	B/D
PAULSON	B	PENNSUCO	D	PHEENEY	C	PINKHAM	B	PLUSH	B
PAULVILLE	B	PENNY	D	PHELAN	D	PINKSTON	B	PLUTOS	B
PAUMALU	B	PENO	C	PHELPS	B	PINNACLES	C	PLYMOUTH	A
PAUNSAUGUNT	D	PENOYER	B	PHERSON	B	PINNEBOG	A/D	POARCH	B
PAUSANT	B	PENROSE	D	PHIFERSON	B	PINO	C	POBER	C
PAUWELA	B	PENSORE	D	PHILBON	D	PINOLE	B	POCALLA	A
PAVAIAI	C	PENTHOUSE	D	PHILDER	D	PINON	D	POCAN	B
PAVANT	D	PENTZ	D	PHILIPPA	C	PINONES	D	POCASSET	B
PAVILLION	B	PENWELL	A	PHILIPSBURG	B	PINTAS	B	POCATELLO	B
PAVO	B	PENWOOD	A	PHILLIPS	C	PINTLAR	B	POCATY	D
PAVOHROO	B	PEOGA	C	PHILO	B	PINTO	C	POCKER	D
PAWCATUCK	D	PEOH	D	PHILOMATH	D	PINTURA	A	POCOLA	D
PAWHUSKA	D	PEOH, DRAINED	C	PHING	D	PINTWATER	D	POCOMOKE	B/D
PAWLING	B	PEONE	D	PHIPPS	C	PIOPOLIS	C/D	POCONO	B
PAWNEE	D	PEONE, DRAINED	C	PHOEBE	B	PIPELINE	D	PODEN	B
PAXTON	C	PEORIA	D	PHOENIX	D	PIPER	C	PODMOR	C
PAXVILLE	B/D	PEOTONE	B/D	PHYS	B	PIPESTONE	B	PODO	D
PAYETTE	B	PEPAL	B	PIASA	D	PIRO	B	PODUNK	B
PAYMASTER	B	PEPOON	D	PIBLER	D	PIRO	A	POE	C
PAYNE	C	PEPPER	D	PICABO	C	PIROUETTE	D	POGAL	C
PAYNECREEK	B	PEQUANING	A	PICACHO	C	PIRUM	B	POGANEAB	C
PAYSON	D	PEQUEA	B	PICANTE	D	PISGAH	C	POGANEAB, CLAYEY	D
PAZAR	B	PERAZZO	B	PICAYUNE	B	PISHKUN	B	SUBSTRATUM	
PEACHAM	D	PERCETON	B	PICEANCE	C	PISMO	D	POGANEAB, SALINE,	C
PEARL	B	PERCHAS	D	PICKAWAY	C	PIT	D	DRAINED	
PEARL HARBOR	D	PERCILLA	D	PICKENS	D	PITCHER	B	POGANEAB, SALINE	D
PEARSOLL	D	PERCIVAL	C	PICKETT	C	PITCO	D	POGANEAB, HIGH	D
PEASLEY	D	PERCOUN	C	PICKFORD	D	PITNEY	C	RAINFALL	
PEAVINE	C	PERCY	B/D	PICKNEY	A/D	PITTMAN	C	POGANEAB, STRONGLY	D
PEAWICK	D	PERDIN	C	PICKRELL	D	PITTSFIELD	B	SALINE	
PEBBLEPOINT	C	PERELLA	B/D	PICKTON	A	PITTSTOWN	C	POGANEAB,	D
PECATONICA	B	PERELLA,	B	PICKUP	D	PITZER	C	SALINE-ALKALI	
PECKISH	D	MODERATELY WET		PICKWICK	B	PIUTE	D	POGANEAB, DRAINED	C
PECOS	D	PERHAM	B	PICO	B	PIVOT	A	POGANEAB, SANDY	C
PECTURE	B	PERICO	B	PICOSA	C	PIXLEY	C	SUBSTRATUM	
PEDEE	C	PERIDGE	B	PIDINEEN	D	PIZENE	B	POGUE	B
PEDERNALES	C	PERITSA	C	PIE CREEK	D	PLACEDO	D	POHAKUPU	B
PEDIGO	B	PERKINS	C	PIERIAN	B	PLACENTIA	D	POIN	D
PEDLEFORD	C	PERKS	A	PIERPONT	C	PLACERITOS,	B	POINDEXTER	B
PEDOLI	B	PERLA	C	PIERRE	D	SALINE, DRAINED		POINSETT	B
PEDRICK	B	PERMA	B	PIERSONTE	A	PLACERITOS,	C	POINT	C
PEDRO	C	PERN	B	PIETOWN	B	SALINE-ALKALI		POINT ISABEL	C
PEEBLES	C	PERNITAS	C	PIIHONUA	A	PLACERITOS,	B	POISONCREEK	D
PEEL	C	PERNTY	D	PIKE	B	MODERATELY WET		POJOAQUE	B
PEELER	B	PERQUIMANS	D	PIKEVILLE	B	PLACERITOS, WET	C	POKEGEMA	B
PEETZ	A	PERRIN	B	PILCHUCK	C	PLACERITOS,	B	POKEMAN	C
PEEVER	C	PERRINE	D	PILEUP	B	DRAINED		POKER	C
PEEVYWELL	C	PERRINTON	C	PILINE	D	PLACID	B/D	POKEY	C

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TABLE 7.1--HYDROLOGIC GROUPS OF THE SOILS OF THE UNITED STATES

POLALLIE	C	POTH	C	PUGSLEY	C	QUINLAN	C	RANDSBURG	D
POLAR	B	POTLATCH	C	PUHI	B	QUINN	B/D	RANGER	C
POLATIS	C	POTOMAC	A	PUHIMAU	D	QUINNEY	C	RANRUFF	D
POLAWANA	A/D	POTOSI	A	PULA	C	QUINTANA	B	RANSLO	D
POLECREEK	D	POTRATZ	C	PULASKI	B	QUINTON	C	RANSOM	B
POLELINE	B	POTSDAM	C	PULCAN	C	QUITMAN	C	RANSTEIN	B
POLEPATCH	A	POTTER	C	PULEHU	B	QUIVERA	C	RANTOUL	D
POLERUN	B	POTTINGER	B	PULEXAS	B	QUONSET	A	RAPATEE	D
POLEY	C	POTTS	B	PULLMAN	D	QUOSATANA	C	RAPELJE	B
POLICH	C	POTTSBURG	B/D	PULPIT	C	RABBITEX	B	RAPH	B
POLKING	D	POUDRE	C	PULS	D	RABER	C	RAPHO	B
POLLARD	C	POULSBO	C	PULSIPHER	D	RABIDEUX	B	RAPIDAN	B
POLLUX	C	POUNCEY	D	PULTNEY	C	RABUN	B	RAPLEE	C
POLLY	B	POVERTY	D	PUMEL	D	RACE	B	RAPPAHANNOCK	D
POLO, MODERATELY	C	POVEY	B	PUMPER	B	RACHERT	D	RAPSON	B
SLOW PERM		POWDER	B	PUNA	A	RACINE	B	RARDEN	C
POLO, MODERATE	B	POWDERHORN	C	PUNALUU	D	RACKER	A	RARICK	C
PERMEABILITY		POWELL	C	PUNCHBOWL	D	RACOMBES	B	RARITAN	C
POLONIO	B	POWER	B	PUNOIP	D	RACOON	C/D	RASBAND	B
POLSON	B	POWLEY	D	PUNG	C	RAD	B	RASILLE	B
POMAT	C	POY	D	PUNGO	D	RAD, ALKALI	B	RASSER	B
POMELLO	C	POYGAN	D	PUNOHU	A	RAD, LACUSTRINE	B	RASSET	B
POMFRET	A	POYNOR	B	PURCELLA	B	SUBSTRATUM		RATAKE	D
POMO	B	POZO	C	PURCHES	C	RAD, FLOODED	C	RATHBUN	C
POMONA	B/D	POZO BLANCO	B	PURDAM	C	RADDLE	B	RATHDRUM	B
POMONA,	D	PRAG	C	PURDY	D	RADER	D	RATLIFF	B
DEPRESSIONAL		PRAIRIEVILLE	B	PURETT	B	RADERSBURG	B	RATON	D
POMPANO	A/D	PRATHER	C	PURGATORY	C	RADFORD	B	RATSOW	C
POMPANO,	A/D	PRATLEY	C	PURNER	D	RADLEY	B	RATTLER	D
DEPRESSIONAL		PRATT	A	PURSLEY	B	RADNOR	C	RATTO	C
POMPANO, FLOODED	D	PREACHER	B	PURVES	D	RAFAEL	D	RATTO, STONY	D
POMPTON	B	PREAKNESS	B/D	PUSHMATAHA	C	RAFTON	C/D	RAUB	C
POMROY	C	PREATORSON	B	PUSTOI	B	RAGLAN	B	RAUGHT	B
PONCA	B	PREBISH	C/D	PUTNAM	D	RAGNAR	A	RAUVILLE	D
PONCENA	D	PREBLE	C	PUTNEY	B	RAGO	C	RAUZI	B
PONCHA	A	PRELO	B	PUTT	C	RAGSDALE	B/D	RAVALLI	D
POND	D	PREMIER	B	PUU OO	A	RAGTOWN	C	RAVEN	A
POND CREEK	B	PRENTISS	C	PUU OPAE	B	RAHAL	C	RAVENDALE	D
PONDER	D	PRESA	B	PUU PA	A	RAHM	C	RAVENELL	D
PONIL	D	PRESHER	B	PUU PA, NONSTONY	B	RAHWORTH	B	RAVENNA	C
PONINA	D	PRESTO	B	PUUKALA	D	RAIL	C/D	RAVENSWOOD	C
PONTO	B	PRESTON	A	PUUONE	C	RAILCITY	A	RAVIA	C
PONTOTOC	B	PREWITT	B	PUYALLUP	B	RAINBOW	C	RAVOLA	B
PONZER	D	PREY	C	PYLE	C	RAINEY	C	RAWAH	C
POOKU	B	PRICE	B	PYLON	D	RAINIER	C	RAWE	C
POOLER	D	PRIDA	C	PYOTE	A	RAINO	D	RAWLES	B
POOLEVILLE	C	PRIDHAM	D	PYRAMID	D	RAINS	B/D	RAWSON	B
POORCAL	B	PRIESTLAKE	B	PYRMONT	D	RAINSBORO	C	RAYBURN	D
POORMA	B	PRIETA	D	PYWELL	D	RAIRDENT	B	RAYEX	D
POOSE	D	PRIM	D	QUAFENO	C	RAISIO	C	RAYFORD	C
POOTATUCK	B	PRIMEAUX	C	QUAKER	C	RAKE	D	RAYMONDVILLE	D
POPASH	D	PRIMEN	D	QUAKERTOWN	C	RAKIED	C	RAYNE	B
POPE	B	PRIMGHAR	B	QUAM	B/D	RALEIGH	D	RAYNESFORD	B
POPLE	C/D	PRINCETON	B	QUAMON	A	RALLOD	D	RAYNHAM	C
POPLIMENTO	C	PRINEVILLE	C	QUANAH	B	RALLS	B	RAYNOLDSON	B
POPPLETON	A	PRING	B	QUANDER	B	RALPH	B	RAYPOL	C
POQUITA	B	PRINGLE	C	QUANTICO	B	RALPHSTON	B	RAZITO	A
POQUONOCK	C	PRITCHETT	C	QUARLES	D	RAMADERO	B	RAZOR	C
PORFIRIO	C	PROCHASKA	A/D	QUARTZBURG	C	RAMBLA	C	RAZORBA	B
PORRETT	D	PROCTOR	B	QUARTZVILLE	B	RAMELLI	D	RAZORT	B
PORT	B	PROGRESSO	C	QUARZ	C	RAMIRES	C	READING	B
PORT BYRON	B	PROMISE	D	QUATAMA	C	RAMIRES, COBBLY	C	READINGTON	C
PORTAGE	D	PROMO	D	QUAY	B	SUBSTRATUM		READLYN	B
PORTAGEVILLE	D	PRONG	C	QUAZO	D	RAMIRES, STONY	C	REAGAN	B
PORTALES	B	PROSPECT	B	QUEALMAN	B	RAMIRES,	D	REAKOR	B
PORTALTO	B	PROSPER	B	QUEALY	D	NONGRAVELLY		REAL	D
PORTERFIELD	C	PROSSER	C	QUEBRADA	C	RAMMEL	C	REAP	D
PORTERS	B	PROTIVIN	C	QUEENY	D	RAMO	C	REARDAN	C
PORTERVILLE	D	PROUT	C	QUEETS	B	RAMONA	B	REAVILLE	C
PORTHILL	D	PROUTY	C	QUEMADO	C	RAMONA, GRAVELLY,	B	REBA	C
PORTIA	C	PROVIDENCE	C	QUENZER	D	COOL		REBEL	B
PORTINO	C	PROVO	D	QUERC	C	RAMONA, COOL	B	RED BAY	B
PORTLAND	D	PROVO BAY	D	QUERENCIA	B	RAMONA, HARD	C	RED BLUFF	C
PORTMOUNT	B	PROW	D	QUETICO	D	SUBSTRATUM		RED BUTTE	B
PORTNEUF	B	PRUE	B	QUICKSELL	C	RAMPART	B	RED HOOK	C
PORTOLA	B	PRUITTON	B	QUIDEN	B	RAMPARTER	B	RED ROCK	B
PORTSMOUTH	B/D	PRUNIE	D	QUIETUS	C	RAMPS	B	RED SPUR	B
PORUM	D	PRYOR	C	QUIGLEY	B	RAMROD	C	REDBANK	B
POSANT	C	PTARMIGAN	C	QUIHI	C	RAMSDELL	C	REDBELL	B
POSEY	B	PUAPUA	D	QUILCENE	C	RAMSEY	D	REDBY	B
POSEYVILLE	C	PUAULU	A	QUILLAYUTE	B	RAMSHORN	B	REDCAN	D
POSITAS	D	PUCHYAN	B	QUILOTOSA	D	RANCE	C	REDCHIEF	C
POSKIN	C	PUERCO	D	QUILT	D	RANDADO	C	REDCLIFF	C
POSOS	C	PUERTA	D	QUIMA	B	RANDALL	D	REDCLOUD	B
POST	D	PUETT	D	QUINCY	A	RANDCORE	D	REDCO	D
POTCHUB	C	PUFFER	D	QUINCY, WET	B	RANDMAN	D	REDCREEK	D
POTEET	C	PUGET	D	QUINCY, GRAVELLY	A	RANDOLPH	C	REDDICK	B/D
POTELL	B	PUGET, DRAINED	C	SUBSTRATUM		RANDS	C	REDDING	D

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REDFEATHER	D	RENSHAW	B	RIDGEPORT	B	ROBOZO	C	ROSE VALLEY	C
REDFIELD	B	RENSLOW	B	RIDGEVIEW	D	ROBROOST	B	ROSEBERRY	D
REDFIELD, WET	C	RENSSELAER	B/D	RIDGEVILLE	B	ROBSON	D	ROSEBLOOM	D
REDIG	B	RENSSELAER, TILL	B/D	RIDIT	C	ROBY	C	ROSEBUD	B
REDLAKE	D	SUBSTRATUM		RIDOTT	C	ROCA	C	ROSEBURG	B
REDLANDS	B	RENSSELAER,	B/D	RIEDEL	C	ROCA, GRAVELLY	D	ROSEDHU	B/D
REDLODGE	D	BEDROCK		RIEDTOWN	C	ROCHE	D	ROSEGLEN	B
REDMANSON	B	SUBSTRATUM		RIESEL	C	ROCHELLE	C	ROSEHAVEN	B
REDMOND	C	RENSSELAER, SANDY	B/D	RIETBROCK	C	ROCHESTER	A	ROSEHILL	D
REDNIK	B	SUBSTRATUM		RIFLE	A/D	ROCIO	C	ROSELAND	B
REDNUN	C	RENSSELAER, CLAY	C	RIGA	D	ROCK CREEK	D	ROSELLA	D
REDOLA	B	LOAM SUBSTRATUM		RIGGINS	D	ROCK RIVER	B	ROSELMS	D
REDONA	B	RENTILL	B	RIGGS	D	ROCKAWAY	C	ROSENWALL	D
REDONDO	B	RENTON	D	RIGLEY	B	ROCKBRIDGE	B	ROSEVILLE	B
REDPORT	B	RENTON, DRAINED	C	RIGOLETTE	C	ROCKCASTLE	D	ROSEWOOD	A/D
REDRIDGE	B	RENTSAC	D	RILEY	B	ROCKDALE	A	ROSEWOOD, WET	D
REDROB	D	RENTZEL	C	RILLA	B	ROCKERS	C	ROSHE SPRINGS	D
REDSPRINGS	B	REPARADA	D	RILLINO	B	ROCKERVILLE	B	ROSHE SPRINGS,	C
REDSPRINGS, GRADED	D	REPP	D	RILLITO	B	ROCKFORD	B	DRAINED	
REDSIDE	B	REPPART	B	RIMER	C	ROCKHOUSE	A	ROSHE SPRINGS,	D
REDSTONE	A	REPUBLIC	B	RIMINI	A	ROCKINCHAIR	C	CLAY SUBSTRATUM	
REDTHAYNE	B	RESCUE	B	RIMROCK	D	ROCKLIN	D	ROSHE SPRINGS,	D
REDTOM	B	RESNER	B	RIMTON	C	ROCKLY	D	VERY POORLY	
REDVALE	C	RESORT	D	RIN	B	ROCKOA	B	DRAINED	
REDVIEW	B	RESOTA	A	RINCON	C	ROCKTON	B	ROSHOLT	B
REDVIEW, WET	C	RESTON	C	RINDA	D	ROCKWELL	B/D	ROSITAS	A
REDWASH	D	RET	D	RINDGE	D	ROCKWOOD	C	ROSITAS, WET	C
REE	B	RETRIEVER	D	RINDGE, DRAINED	C	ROCKY FORD	B	ROSITAS, GRAVELLY	A
REEBOK	D	RETROP	C	RINEARSON	B	ROCKYBAR	B	ROSITAS, LOAMY	A
REED	D	REVA	D	RINEY	B	RODELL	D	ROSLYN	B
REED, DRAINED	C	REVEL	C	RING	C	RODEO	D	ROSMAN	B
REEDER	B	REVENTON	B	RINGLE	B	RODESSA	D	ROSNEY	B
REEDSBURG	C	REVERE	B/D	RINGLING	A	RODMAN	A	ROSS	B
REEDSPORT	C	REWARD	B	RINGO	D	ROEBUCK	D	ROSSBURG	B
REEDY	D	REXBURG	B	RINGWOOD	B	ROELLEN	D	ROSSFIELD	B
REEFRIDGE	D	REXFORD	C	RINKER	C	ROEMER	C	ROSSMOYNE	C
REELFOOT	C	REXMONT	D	RIO	D	ROETEX	D	ROSWELL	A
REESE	C	REXOR	A	RIO ARRIBA	D	ROFISS	B	ROSY	B
REESVILLE	C	REYAB	B	RIO DIABLO	C	ROGAN	B	ROTAMER	B
REEVES	B	REYES	D	RIO GRANDE	B	ROGERSON	D	ROTAN	C
REFLECTION	B	REYNOSA	B	RIO LAJAS	A	ROGERT	D	ROTHIEMAY	B
REFUGE	C	REYWAT	D	RIO PIEDRAS	B	ROGUE	B	ROTHIEMAY, STONY	C
REGAL	B/D	REZAVE	D	RIOCONCHO	C	ROHNERVILLE	B	ROTHIEMAY, WET	D
REGAN	B/D	RHAME	B	RION	B	ROHRERSVILLE	D	ROTHSAY	B
REGENT	C	RHEA	B	RIPLEY	B	ROIC	D	ROTINOM	B
REGGAD	A	RHINEBECK	D	RIPON	B	ROLETTE	C	ROTO	B
REGGEAR	C	RHOADES	D	RIPPLE	B	ROLFE	C	ROTTULEE	C
REHBURG	C	RHOAME	C	RIPPOWAN	C	ROLISS	B/D	ROUBIDEAU	C
REHFIELD	C	RHOAMETT	C	RIRIE	B	ROLLA	C	ROUEN	C
REHFIELD	B	RHONE	B	RISBECK	B	ROLLINGSTONE	C	ROUGHCREEK	D
REHM	C	RIB	B/D	RISLEY	D	ROLOC	D	ROUGHMOUNT	C
REICHEL	B	RIBERA	C	RISUE	D	ROLOFF	C	ROUND BUTTE	D
REIFF	B	RIBHILL	B	RITA	D	ROMBERG	B	ROUNDOR	C
REILLY	A	RICCO	D	RITCHEY	D	ROMBO	C	ROUNDTOP	C
REINA	D	RICEBORO	B/D	RITNER	C	ROME	B	ROUNDUP	C
REINACH	B	RICERT	B	RITO	B	ROMEO	D	ROUNDY	C
REINER	B	RICETON	B	RITTER	B	ROMERO	D	ROUSSEAU	A
REKOP	D	RICEVILLE	C	RITTMAN	C	ROMGAN	C	ROUTON	D
RELAN	B	RICH	C	RITZ	D	ROMIA	B	ROUTT	C
RELAY	B	RICH, WET	D	RITZ, DRAINED	C	RONINE	B	ROVAL	D
RELIANCE	C	RICHARDSON	B	RITZCAL	B	RONINELL	C	ROWDEN	C
RELIZ	D	RICHENS	C	RITZVILLE	B	RONNELL	B/D	ROWDY	B
RELLEY	B	RICHEY	C	RIVERDALE	A	RONULUS	D	ROWE	D
RELSOB	B	RICHFIELD	B	RIVERHEAD	B	RONAN	D	ROWEL	D
RELUCTAN	C	RICHFORD	A	RIVERSIDE	A	RONO	C	ROWENA	C
REMBERT	D	RICHLAND	B	RIVERTON	B	RONDEAU	A/D	ROWLAND	C
REMLAP	C	RICHMOND	D	RIVERVIEW	B	RONNEBY	C	ROWLEY	C
REMLIK	A	RICHTER	B	RIVIERA	C/D	RONSEL	B	ROXAL	D
REMMIT	B	RICHVALE	B	RIVIERA,	D	RONSON	B	ROXANA	B
REMNOY	D	RICHVIEW	C	DEPRESSIONAL		ROONEY	D	ROXBURY	B
REMOTE	B	RICHVILLE	C	RIVRA	A	ROOSET	B	ROXER	B
REMSEN	D	RICHWOOD	B	RIXIE	C	ROOT	B/D	ROXTON	D
REMUNDA	C	RICKER	D	RIZ	D	ROOTEL	C	ROY	B
REMUS	B	RICKNAN	C	RIZOZO	D	ROPER	B/D	ROYAL	B
RENBAC	D	RICKMORE	C	ROANE	C	ROSALIE	B	ROYCE	C
RENCALSON	C	RICKREALL	D	ROANHIDE	C	ROSAMOND	B	ROYOSA	A
RENCOT	D	RICKS	A	ROANOKE	D	ROSAMOND,	C	ROYST	C
RENFROW	D	RICOT	C	ROARING	B	SALINE-ALKALI		ROYSTONE	B
RENICK	D	RICREST	B	ROB ROY	C	ROSAMOND, SANDY	C	ROZA	C
RENISH	C	RIDD	C	ROBANA	B	SUBSTRATUM		ROZELLVILLE	B
RENNER	B	RIDDLES	B	ROBBS	D	ROSANE	B/D	ROZETTA	B
RENNIE	D	RIDENBAUGH	D	ROBCO	C	ROSANKY	C	ROZLEE	C
RENNIE, DRAINED	C	RIDGE	B	ROBER	B	ROSARIO	C	RUARK	B/D
RENO	D	RIDGEBURY	C	ROBERTSDALE	C	ROSCOE	D	RUBICON	A
RENOHILL	C	RIDGECREST	C	ROBERTSVILLE	D	ROSCOMMON	A/D	RUBIO	C/D
RENOL	C	RIDGEDALE	B	ROBIN	B	ROSE CREEK	C	RUBSON	B
RENOVA	B	RIDGELAND	B/D	ROBINETTE	B	ROSE CREEK,	B	RUBY	B
RENOX	B	RIDGELAWN	B	ROBINSONVILLE	B	DRAINED		RUBYHILL	C

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RUCH	B	SAGERTON	C	SANGER	D	SAWBUCK	B	SEANAN	B
RUCKER	B	SAGLE	C	SANGO	C	SAWCREEK	C	SEAQUEST	C
RUCKLES	D	SAGO	D	SANHEDRIN	B	SAWMILL	B/D	SEAR	B
RUCLICK	C	SAGOUSPE	C	SANIBEL	B/D	SAWTELL	C	SEARING	B
RUDD	D	SAGUACHE	B	SANILAC	B	SAWTELPEAK	D	SEARLA	B
RUDOLEY	D	SAL	D	SANJE	B	SAWYER	C	SEARLES	C
RUDEEN	C	SALADAR	D	SANLOREN	B	SAXBY	D	SEARSPORT	D
RUDYARD	D	SALADON	D	SANPETE	B	SAXON	B	SEARSVILLE	D
RUELLA	B	SALAL	C	SANPITCH	C	SAY	B	SEASTRAND	D
RUFUS	D	SALAMATOF	D	SANSARC	D	SAYBROOK	B	SEATON	B
RUGLES	B	SALANDER	B	SANTA	D	SAYDAB	C	SEATTLE	D
RUHE	D	SALAS	C	SANTA CLARA	C	SAYERS	A	SEATTLE, DRAINED	C
RUIDOSO	C	SALCHAKET	B	SANTA FE	D	SAYLES	D	SEAVERSON	D
RUKO	D	SALCO	B	SANTA LUCIA	C	SAYLESVILLE	C	SEAWILLOW	B
RULE	B	SALEN	B	SANTA MARTA	C	SAYNER	A	SEBAGO	D
RUMBLECREEK		SALERATUS	C	SANTA YNEZ	D	SCALA	B	SEBASTIAN	D
RUMBO	C	SALERNO	B/D	SANTANA	D	SCALLEY	B	SEBASTOPOL	C
RUMFORD	B	SALGA	C	SANTANELA	D	SCAMMAN	D	SEBEWA	B/D
RUMNEY	C	SALIDA	A	SANTAROSA	B	SCANDARD	C	SEBREE	D
RUMPLE	C	SALINAS	B	SANTEE	D	SCANTIC	D	SEBRING	B/D
RUMUNG	C	SALISBURY	D	SANTIAGO	B	SCAPONIA	B	SEBUD	B
RUNE	C	SALIX	B	SANTIAN	C	SCAR	B	SECCA	C
RUNEBERG	C/D	SALKUM	C	SANTO	B	SCARBORO	D	SECESH	B
RUNGE	B	SALLISAW	B	SANTO TOMAS	B	SCATLAKE	D	SECRET CREEK	B
RUNN	D	SALLYANN	C	SANTONI	D	SCAVE	C	SEDALE	D
RUPLEY	A	SALMO	C/D	SAPELO	D	SCHAFFENAKER	A	SEDGEFIELD	C
RUSCO	C	SALMON	B	SAPINERO	B	SCHAMBER	A	SEDGWICK	B
RUSCO, PONDED	D	SALONIE	D	SAPKIN	C	SCHAMP	C	SEDILLO	B
RUSE	D	SALT CHUCK	A	SAPPHIRE	B	SCHAPVILLE	C	SEDMAR	D
RUSH	B	SALT LAKE	D	SAPPINGTON	B	SCHAWANA	D	SEDWELL	C
RUSHFORD	B	SALTAIR	D	SARA	D	SCHENCO	D	SEEDSKADEE	D
RUSHMORE	B/D	SALTER	B	SARAGOSA	B	SCHERRARD	D	SEELEZ	B
RUSHTOWN	A	SALTERY	D	SARALEGUI	B	SCHLEY	B	SEELOVERS	C
RUSHVILLE	D	SALTESE	D	SARANAC	C/D	SCHMUTZ	B	SEELYEVILLE	A/D
RUSO	B	SALTESE, DRAINED	C	SARANAC, GRAVELLY	C	SCHNEBLY	D	SEELYEVILLE,	D
RUSON	C	SALTINE	C	SUBSTRATUM		SCHNEIDER	B	SLOPING	
RUSSELL	B	SALTON	D	SARATON	C	SCHNIPPER	C	SEEPRID	B
RUSSIAN	B	SALUDA	C	SARBEN	B	SCHNOORSON	C	SEES	C
RUSSLER	C	SALVISA	C	SARDINIA	C	SCHNORBUSH	B	SEEWEE	B
RUSTICO	B	SALZER	D	SAROIS	C	SCHODSON	C	SEGIDAL	D
RUSTIGATE	C	SALZER, DRAINED	C	SARGEANT	D	SCHOFIELD	C	SEGNO	C
RUSTON	B	SAMBA	D	SARITA	A	SCHOHARIE	C	SEGUIN	B
RUSTY	B	SAMBRITO	B	SARKAR	D	SCHOLLE	B	SEGURA	D
RUTAB	B	SAMISH	D	SARNOSA	B	SCHOODIC	D	SEHONE	C
RUTHERFORD	C	SAMISH, DRAINED	C	SARONA	B	SCHOOLCRAFT	B	SEHORN	D
RUTLAND	C	SAMMAMISH	D	SARPY	A	SCHOOLEY	D	SEIS	C
RUTLEGE	B/D	SAMMAMISH, DRAINED	C	SARTELL	A	SCHOOLEY, DRAINED	C	SEITZ	C
RYAN	D	SAMPSEL	D	SASKA	B	SCHOOLHOUSE	D	SEJITA	D
RYAN PARK	B	SAMPSON	B	SASPAMCO	B	SCHOONER	D	SEKIL	B
RYARK	A	SAMSIL	D	SASSAFRAS	B	SCHRADER	C	SEKIU	D
RYDE	C	SAMSULA	B/D	SASSER	B	SCHRAP	D	SELAH	C
RYDER	C	SAN ANDREAS	B	SATANKA	C	SCHRIER	B	SELDEN	C
RYDOLPH	C	SAN ANTON	B	SATANTA	B	SCHROCK	B	SELEVIN	D
RYEGATE	C	SAN ANTONIO	C	SATATTON	D	SCHULINE	B	SELFRIDGE	B
RYELL	B	SAN ARCACIO	B	SATELLITE	A	SCHUMACHER	B	SELIA	C
RYEPATCH	D	SAN ARCACIO,	C	SATILLA	D	SCHUSTER	B	SELIGMAN	D
RYER	C	SALINE		SATIN	C	SCHUYLER	B	SELKIRK	C
RYKER	B	SAN BENITO	B	SATSOP	B	SCIO	B	SELLE	B
RYORP	C	SAN EMIGDIO	B	SATTLEY	B	SCIOTOVILLE	C	SELLERS	B/D
RYPOD	B	SAN GERMAN	D	SATTRE	B	SCISM	C	SELMA	B/D
RYUS	B	SAN ISABEL	A	SATURN	B	SCITICO	C	SELMAC	D
SAAR	C	SAN JOAQUIN	D	SATUS	B	SCITUATE	C	SELON	
SABANA	D	SAN JON	C	SAUCEL	D	SCOAP	B	SELWAY	B
SABANA SECA	D	SAN JOSE	B	SAUCIER	C	SCOBEY	C	SEMIAHMOO	D
SABE	B	SAN JUAN	A	SAUDE	B	SCOGGIN	D	SEMIAHMOO, DRAINED	C
SABENYO	B	SAN LUIS	C	SAUGATUCK	C	SCOON	D	SEMINOLE	D
SABINA	C	SAN MATEO	C	SAUGUS	B	SCOOTENEY	B	SEN	B
SABLE	B/D	SAN MIGUEL	D	SAUK	B	SCORUP	C	SENCHERT	C
SAC	B	SAN SABA	D	SAULICH	D	SCOTCO	A	SENECAVILLE	B
SACHEEN	A	SAN SEBASTIAN	B	SAUM	C	SCOTIA	B	SENSABAUGH	B
SACHETT	C	SAN SIMEON	D	SAUNDERS	D	SCOTT	D	SEQUATCHIE	B
SACO	D	SAN TIMOTEO	B	SAURIN	C	SCOTT LAKE	B	SEQUIM	A
SACRAMENTO	D	SAN TIMOTEO,	C	SAUVIE	C/D	SCOTTIES	B	SEQUOIA	C
SACUL	C	GRAVELLY		SAUVIE, MODERATELY	C	SCOUT	B	SERDEN	A
SADDLE	C	SAN YSIDRO	D	WET		SCRABBLERS	A	SERENE	C
SADER	D	SANCHEZ	D	SAUVIE, PROTECTED	C/D	SCRANTON	A/D	SEROCO	A
SADIE	C	SANDALL	C	SAUVOLA	C	SCRAVO	B	SERPA	D
SADLER	C	SANDERSON	B	SAUZ	B	SCRIBA	C	SERPEN	B
SAFFELL	B	SANDHILL	B	SAVAGE	C	SCRIVER	B	SERPENTANO	B
SAG	B	SANDIA	B	SAVAGE, SALINE	D	SCROGGIN	C	SERPOD	C
SAGANING	A/D	SANDOVAL	D	SAVAGE, PE>31	C	SCULLIN	C	SERRANO	D
SAGASER	B	SANDOVAL, DRY	B	SAVANNAH	C	SCUPPERNONG	D	SERVILLETA	D
SAGE	D	SANDRIDGE	A	SAVENAC	C	SEABROOK	C	SESANE	C
SAGECREEK	B	SANDUN	B	SAVO	C	SEAFIELD	B	SESPE	C
SAGEDALE	C	SANDWASH	C	SAVOIA	B	SEAFORTH	B	SESSIONS	C
SAGEHILL	B	SANDWICK	B	SAVONIA	B	SEAGATE	A/D	SESSUM	D
SAGEMOOR	B	SANELI	D	SAWABE	D	SEAGOVILLE	D	SET	C
SAGERS	B	SANFORD	B	SAWATCH	B/D	SEALY	B	SETH	C

NOTES: TWO HYDROLOGIC SOIL GROUPS SUCH AS B/C INDICATES THE DRAINED/UNDRAINED SITUATION.
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TABLE 7.1--HYDROLOGIC GROUPS OF THE SOILS OF THE UNITED STATES

SETTERS	C	SHELBURNE	C	SHOWALTER,	D	SIOUX	A	SMYRNA	A/D
SETTLEMENT	D	SHELBY	B	GRAVELLY		SIPPLE	B	SNAG	B
SETTLEMEYER	D	SHELBYVILLE	B	SHOWLOW	C	SIRDRAK	A	SNAHOPISH	B
SETTLEMEYER,	D	SHELD	B	SHREE	B	SIRI	B	SNAKE	C
SALINE-ALKALI		SHELL	B	SHREWDER	B	SIROCO	C	SNAKE HOLLOW	B
SETTLEMEYER,	D	SHELLABARGER	B	SHREWSBURY	C/D	SIRRETTA	C	SNAKELUM	B
MODERATELY WET		SHELLBLUFF	B	SHRINE	B	SISKIYOU	B	SNAPP	C
SETTLEMEYER,	D	SHELLORAKE	A	SHROUTS	D	SISSETON	B	SNEAD	D
DRAINED		SHELLROCK	A	SHUBUTA	C	SISSON	B	SNEFFELS	C
SETTLEMEYER,	D	SHELMADINE	D	SHUE	C	SISTERS	A	SNELL	C
FLOODED		SHELOCTA	B	SHUKASH	A	SITES	C	SNELLING	B
SETTLEMEYER, COOL	D	SHELTON	C	SHUKSAN	C	SIWELL	C	SNIDER	C
SETTLEMEYER,	C	SHENA	D	SHULE	C	SIXBEACON	B	SNOHOMISH	D
RARELY FLOODED		SHENANDOAH	D	SHULLSBURG	C	SIXMILE	C	SNOHOMISH, DRAINED	C
SEVENMILE	B	SHENKS	B/D	SHUMLA	C	SIZER	B	SNOMO	C
SEVERN	B	SHENVAL	B	SHUMWAY	D	SKAGGS	C	SNOOK	D
SEVILLE	D	SHEP	B	SHUPERT	C	SKAGIT	D	SNOQUALMIE	A
SEVY	B	SHEPAN	C	SHURLEY	A	SKAMA	A	SNOW	B
SEWANEE	B	SHEPPARD	A	SHUSTER	C	SKALAN	C	SNOWLIN	B
SEWARD	B	SHERANDO	B	SI	C	SKAMANIA	B	SNOWMORE	C
SEXTON	C/D	SHERAR	C	SIBELIA	B	SKAMO	C	SNOWSHOE	B
SEYMOUR	D	SHERBURNE	C	SIBLEY	B	SKANEE	C	SNOWSLIDE	B
SHAAK	C	SHERIDAN	B	SIBLEYVILLE	B	SKANID	D	SNOWSTORM	D
SHABLISS	D	SHERLESS	B	SICKLES	B/D	SKATE	B	SNOWVILLE	D
SHACK	B	SHERLOCK	B	SICKLESTEETS	B	SKEDADDLE	D	SNUFFUL	C
SHADELAND	C	SHERM	D	SIDELL	B	SKELLOCK	B	SOAKPAK	B
SHADOW	B	SHERMORE	B	SIDLAKE	C	SKELON	C	SOBAY	B
SHADYGROVE	C	SHERRY	B/D	SIDON	C	SKERRY	C	SOBEGA	C
SHAFFTON	B	SHERRY, STONY	D	SIEBEN	B	SKIDMORE	B	SOBOBA	A
SHAGNASTY	C	SHERRYL	B	SIEBERT	A	SKIPANON	B	SOBOL	C
SHAKAMAK	C	SHERWOOD	B	SIECHE	C	SKIPOPA	D	SOBRANTE	C
SHAKER	C	SHEVLIN	C	SIELO	C	SKIYOU	B	SOCORRO	C
SHAKESPEARE	C	SHIDLER	D	SIEROCLIFF	C	SKOKOMISH	D	SODA	B
SHAKOPEE	C	SHIELDS	C	SIERRA	B	SKOKOMISH, DRAINED	C	SODA LAKE	B
SHALAKE	C	SHIFFER	C	SIERRAVILLE	B	SKOLY	B	SODERVILLE	A
SHALAKO	D	SHILOH	B/D	SIESTA	D	SKOOKUM	C	SODHOUSE	D
SHALBA	D	SHIMA	C	SIFTON	B	SKORO	B	SODUS	C
SHALCAR	D	SHIMMON	C	SIGNAL	C	SKOWHEGAN	B	SOELBERG	B
SHALCAR, DRAINED	C	SHINAKU	D	SIGURD	B	SKULL CREEK	C	SOEN	C
SHALET	D	SHINBARA	D	SIKESTON	B/D	SKUMPAH	D	SOFIA	C/D
SHALONA	B	SHINDLER	C	SILAS	B	SKUTUM	C	SOFTSCRABBLE	C
SHAM	D	SHINER	C	SILAS, WET	C	SKYBERG	C	SOGI	C
SHAMBO	B	SHINGLE	D	SILAS, FLOODED	B	SKYHAVEN	C	SOGN	D
SHAMEL	B	SHINGLETOWN	C	SILAWA	D	SKYHIGH	C	SOGO	B
SHAMOCK	C	SHINKEE	C	SILER	B	SKYKOMISH	A	SOGZIE	B
SHANAHAN	B	SHINROCK	C	SILERTON	B	SKYLICK	B	SOLAK	D
SHANDEP	B/D	SHIOCTON	C	SILI	C	SKYLINE	D	SOLANO	D
SHANE	D	SHIPLEY	B	SILSTID	A	SKYMOR	D	SOLDATNA	B
SHANKLER	A	SHIPLEY,	B	SILVA	C	SKYVILLAGE	D	SOLDIER	C
SHANO	B	STRATIFIED		SILVER	C	SKYWAY	B	SOLDUC	B
SHANTA	B	SUBSTRATUM		SILVER CREEK	D	SLABTOWN	B	SOLEDAD	B
SHARATIN	B	SHIPLEY,	C	SILVERADO	B	SLAGLE	C	SOLIER	D
SHARKEY	D	SALINE-ALKALI		SILVERBOW	D	SLAUGHTER	C	SOLIS	C
SHARLAND	B	SHIPLEY,	B	SILVERCHIEF	C	SLAVEN	C	SOLLEKS	C
SHARON	B	NONFLOODED		SILVERCLIFF	B	SLAW	C	SOLLER	D
SHARPS	C	SHIPLEY, RARELY	B	SILVERDALE	A	SLAYTON	D	SOLOMON	D
SHARPSBURG	B	FLOODED		SILVERN	A	SLEEPER	C	SOLONA	C
SHARROTT	D	SHIPLEY, GRAVELLY	B	SILVERTON	C	SLEETH	C	SOMBORDORO	D
SHARVANA	C	SUBSTRATUM		SILVIES	D	SLICKROCK	B	SOMERS	B
SHASER	B	SHIPROCK	B	SIMAS	C	SLIDELL	D	SOMERVELL	B
SHASKIT	B	SHIPS	D	SIMCOE	C	SLIGHTS	C	SOMSEN	C
SHASTA	B	SHIPSHE	B	SIMEON	A	SLIGTING	C	SONAHNPIL	B
SHATRUCE	C	SHIRK	C	SIMEROI	B	SLIKOK	D	SONOCAN	C
SHATTA	C	SHIVELY	B	SIMMONT	C	SLIMBUTTE	B	SONOITA	B
SHATTUCK	B	SHOALS	C	SIMON	C	SLINGER	B	SONOMA	C
SHAVANO	B	SHOAT	D	SIMON, GRAVELLY	B	SLIPMAN	B	SONOMA, MODERATELY	C
SHAVASH	C	SHOEPEG	C	SUBSTRATUM		SLOAN	B/D	WET, SALINE	
SHAVER	B	SHOESTRING	B	SIMONA	D	SLOCUM	C	SONOMA, SALINE,	B
SHAWA	B	SHOKEN	D	SIMONTON	B	SLUICE	B	DRAINED	
SHAWANO	A	SHONKIN	D	SIMPARK	D	SLY	B	SONOMA, SALINE,	C
SHAWMUT	B	SHOOFLIN	C	SIMPATICO	B	SMACKOUT	B	FLOODED	
SHAY	D	SHOOFLY	D	SIMPSON	C	SMALLCONE	D	SONOMA, SALINE	C
SHAYLA	D	SHOOK	C	SIMS	D	SMARTS	B	SONOMA, MODERATELY	C
SHEAR	C	SHOOKER	C	SINAI	C	SMAUG	B	WET	
SHEAVILLE	D	SHOREEK	C	SINAMOX	B	SMEDLEY	D	SONOMA, DRAINED	B
SHEBANG	D	SHOREWOOD	C	SINCLAIR	C	SMELTER	C	SONOMA, FLOODED	C
SHEBEON	C	SHORIM	C	SINGATSE	D	SMILEY	B/D	SONOMA, NONFLOODED	C
SHEDADO	C	SHORT CREEK	C	SINGERTON	B	SMILEYVILLE	D	SONORA	B
SHEDD	C	SHORTCUT	C	SINGLETREE	C	SMILD	C	SONTAG	D
SHEDHORN	D	SHORTYORK	D	SINGSAAS	B	SMITHBORO	D	SOONAHBE	B
SHEEGE	D	SHOSHONE	D	SINKSON	B	SMITHDALE	B	SOOSAP	C
SHEEP CREEK	C	SHOTGUN	C	SINLOC	D	SMITHNECK	B	SOPER	C
SHEEPCAN	B	SHOTWELL	D	SINLOC, DRAINED	C	SMITHTON	D	SOQUEL	B
SHEEPHEAD	C	SHOUNS	B	SINNICE	A	SMITHVILLE	B	SORENSEN	B
SHEEPROCK	A	SHOWALTER	D	SINNIGAM	D	SMITHWICK	D	SORF	C
SHEEPSCOT	B	SHOWALTER, MOIST	D	SINTON	B	SMOCREEK	C	SORRENTO	B
SHEETIRON	B	SHOWALTER, STONY	B	SINUK	D	SMOKEY	C	SORTER	D
SHEFFIELD	D			SION	B	SMOLAN	C	SORUM	D

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SOTIN	B	SPRUCEDALE	D	STEINBECK	B	STRINGTOWN	B	SURGEM	C
SOUGHE	D	SPUD	C	STEINSBURG	C	STRINGTOWN, GRADED	C	SURGH	B
SOULAJULE	C	SPUKWUSH	B	STEIWER	C	STROLE	C	SURPRISE	B
SOUTHACE	B	SPUR	B	STELLAR	C	STROM	C	SURRENCY	D
SOUTHAM	D	SPURGER	C	STEMILT	C	STROMAL	A	SURRETT	C
SOUTHFORK	D	SPURLOCK	B	STEMLEY	C	STRONGHURST	B	SURVEYORS	B
SOUTHGATE	D	SQUALICUM	B	STEMPLE	B	STROUPE	C	SURVYA	C
SOUTHPORT	B	SQUALLY	B	STENDAL	C	STRYKER	C	SUSANNA	C/D
SOUTHRIDGE	B	SQUAW	B	STEPHEN	C	STUBBS	C	SUSIE CREEK	C
SOUTHWICK	C	SQUAWCREEK	D	STEPHENVILLE	B	STUCKY	B	SUSITNA	B
SOWCAN	B	SQUAWROCK	C	STEPROCK	B	STUKEL	D	SUSQUEHANNA	D
SPAA	D	SQUIRES	C	STEPSTONE	B	STUMBLE	A	SUTA	B
SPACE CITY	A	SRIADA	D	STERLING	B	STUMPP	D	SUTCLIFF	B
SPADE	B	ST. ALBANS	B	STERLINGTON	B	STUNNER	B	SUTHER	C
SPADRA	B	ST. ANTHONY	B	STERRETT	D	STUNTZ	C	SUTHERLIN	C
SPAGER	D	ST. AUGUSTINE	C	STETSON	B	STURGEON	B	SUTKIN	B
SPALDING	D	ST. AUGUSTINE,	B	STETTER	D	STURKIE	B	SUTLEY	B
SPANA	D	ORGANIC		STEUBEN	B	STUTTGART	D	SUTPHEN	D
SPANAWAY	B	SUBSTRATUM		STEUBER	B	STUTZMAN	C	SUTRO	C
SPANEL	D	ST. AUGUSTINE,	C	STEVENS	B	STUTZMAN, WET	D	SUTTLER	B
SPANG	B	CLAYEY SUBSTRATUM		STEVENSON	B	STUTZVILLE	C	SUTTON	B
SPANGLER	C	ST. CHARLES	B	STEWART	D	STYX	B	SUVER	D
SPARANK	D	ST. CLAIR	D	STEWVAL	D	SUBACO	D	SVEA	B
SPARHAM	D	ST. ELMO	A	STICKNEY	C	SUBLETTE	B	SVENSEN	B
SPARKHULE	D	ST. GEORGE	B	STIDHAM	B	SUBLIGNA	B	SVERDRUP	B
SPARMO	B	ST. GEORGE, SALINE	C	STIEN	B	SUCHES	B	SWAGER	C
SPARR	C	ST. GEORGE, WET	D	STIGLER	D	SUDBURY	B	SWAKANE	D
SPARTA	A	ST. IGNACE	D	STILGAR	B	SUDDUTH	C	SWALER	D
SPARTA, SILTY CLAY	B	ST. JOHNS	B/D	STILL	B	SUDWORTH	B	SWANBOY	D
LOAM SUBSTRATUM		ST. JOHNS,	D	STILLMAN	B	SUEPERT	C	SWANDAD	B
SPARTA, LOAMY	A	DEPRESSIONAL		STILLWATER	D	SUEY	B	SWANLAKE	B
SUBSTRATUM		ST. LUCIE	A	STILSKIN	C	SUFFIELD	C	SWANNER	D
SPARTA, BEDROCK	A	ST. MARTIN	C	STILSON	B	SUFFOLK	B	SWANSEA	D
SUBSTRATUM		ST. MARYS	B	STIMSON	D	SUGARBOWL	B	SWANSON	C
SPASPREY	C	ST. ONGE	B	STINGAL	B	SUGARDEE	B	SWANTON	C/D
SPEAKER	C	ST. PAUL	B	STINGDORN	D	SUGARLOAF	B	SWANTOWN	C
SPEARFISH	D	ST. THOMAS	D	STIPE	C	SUGLO	B	SWANVILLE	C
SPEARHEAD	B	ST.HELENS	B	STIRK	D	SUISUN	D	SWANWICK	D
SPEARMAN	B	STABLER	B	STIRRUP	B	SULA	B	SWAPPS	C
SPEARVILLE	C	STADY	B	STIRUM	B/D	SULLIVAN	B	SWARTSWOOD	C
SPECK	D	STAFFORD	C	STIRUM, PONDED	D	SULLY	B	SWARTZ	D
SPEELYAI	D	STAGECOACH	B	STISSING	C	SULOAF	B	SWASEY	D
SPEER	B	STAHL	C	STIVERSVILLE	B	SULPHURA	D	SWASTIKA	C
SPEIGLE	B	STAKE	C	STOCKADE	B/D	SULSAVAR	B	SWAUK	D
SPENARD	D	STALEY	B	STOCKBRIDGE	C	SULTAN	C	SWEATMAN	C
SPENCER	B	STALLINGS	C	STOCKEL	C	SUMAN	B/D	SWEDE	B
SPENLO	B	STAMBAUGH	B	STOCKLAND	B	SUMAS	D	SWEEN	C
SPENS	A	STAMFORD	D	STOCKPEN	D	SUMAS, DRAINED	B	SWEENEY	B
SPERRY	C/D	STAMP	D	STOCKTON	D	SUMATRA	B	SWEET	C
SPEXARTH	B	STAMPEDE	D	STODA	B	SUMINE	B	SWEETAPPLE	B
SPHINX	D	STAN	B	STODICK	D	SUMMERFIELD	D	SWEETGRASS	B
SPICER	B/D	STANDLEY	C	STOHLMAN	D	SUMMERS	B	SWEETWATER	D
SPICERTON	D	STANDUP	B	STOKES	D	SUMMERTON	B	SWEM	C
SPICEWOOD	C	STANEY	D	STOKLY	B	SUMMERVILLE	D	SWENODA	B
SPILLCO	B	STANFIELD	C	STOMAR	C	SUMMIT	C	SWIFT	B
SPILLVILLE	B	STANISLAUS	D	STONEBERGER	D	SUMMITVILLE	C	SWIFT CREEK	B
SPINEKOP	B	STAPALDOP	B	STONEBURG	B	SUMPF	D	SWIFTON	A
SPINEKOP, SALINE	C	STAPLETON	B	STONEHAM	B	SUMTER	C	SWIMLEY	C
SPINEKOP,	C	STAPP	C	STONEHEAD	C	SUNYA	D	SWIMS	B
MODERATELY WET		STARBUCK	D	STONELICK	B	SUN	D	SWINGLER	D
SPINKS	A	STARGO	B	STONER	B	SUNAPEE	B	SWINGLER, ALKALI	B
SPINLIN	C	STARHOPE	D	STONEVILLE	B	SUNBURG	B	SWINGLER,	D
SPIRES	D	STARICHKOF	D	STONO	B/D	SUNBURST	C	SALINE-ALKALI	
SPIRIT	C	STARKEY	C	STONYFORD	D	SUNBURY	B	SWINK	D
SPIRO	B	STARKS	C	STOOKEY	C	SUNCITY	D	SWINOMISH	C
SPIVEY	B	STARLEY	D	STORDEN	B	SUNCOOK	A	SWINT	B
SPLENDORA	C	STARMAN	D	STORLA	B	SUNDANCE	B	SWISBOB	D
SPLITRO	D	STARR	C	STORMITT	B	SUNDAY	A	SWISSHELM	B
SPLITTOP	C	STASER	B	STOTT	C	SUNDELL	B	SWITCHBACK	C
SPOFFORD	D	STATE	B	STOUGH	C	SUNEV	B	SWITZERLAND	B
SPOKANE	B	STATLER	B	STOUT	D	SUNFIELD	B	SWOPE	C
SPOKEL	B	STATZ	D	STOVHO	C	SUNLIGHT	D	SWORMVILLE	C
SPONSELLER	B	STAVE	D	STOWE	C	SUNNYHAY	D	SWYGERT	C
SPOOL	D	STAYTON	D	STOWELL	D	SUNNYSIDE	B	SYBLON	D
SPOONER	C/D	STEARNS	D	STOY	C	SUNRAY	B	SYCAMORE, DRAINED	B
SPOTSYLVANIA	C	STECUM	C	STRABER	C	SUNSET	B	SYCAMORE, FLOODED	C
SPOTTSWOOD	B	STEED	A	STRAHAN	B	SUNSHINE	C	SYCAMORE, CLAY	B
SPRAY	B	STEEDMAN	C	STRAIGHT	C	SUNSWEET	C	SUBSTRATUM	
SPRECKELS	C	STEEDMAN, STONY	D	STRANDQUIST	B/D	SUNUP	D	SYCAN	A
SPRING	C	STEEKEE	C	STRAT	B	SUOMI	C	SYCLE	B
SPRINGDALE	A	STEELE	C	STRATFORD	B	SUP	B	SYENITE	C
SPRINGER	B	STEENS	C	STRATTON	C	SUPAN	B	SYLACAUGA	D
SPRINGERVILLE	D	STEEPLAN	D	STRAW	B	SUPERIOR	D	SYLCO	C
SPRINGFIELD	D	STEEVER	B	STRAWN	B	SUPERSTITION	A	SYLVAN	B
SPRINGMEYER	B	STEFF	C	STREATOR	B/D	SUPERVISOR	C	SYLVESTER	B
SPRINGSTEEN	C	STEGALL	C	STREVELL	B	SUPPLEE	B	SYLVIA	C
SPRINGWATER	C	STEIGER	A	STRICKER	B	SUR	B	SYMCO	C
SPROUL	D	STEINAUER	B	STRICKLAND	C	SURFSIDE	D	SYMERTON	B

NOTES: TWO HYDROLOGIC SOIL GROUPS SUCH AS B/C INDICATES THE DRAINED/UNDRAINED SITUATION.
MODIFIERS SHOWN, E.G., BEDROCK SUBSTRATUM, REFER TO A SPECIFIC SOIL SERIES PHASE FOUND IN SOIL MAP LEGEND.

TABLE 7.1--HYDROLOGIC GROUPS OF THE SOILS OF THE UNITED STATES

SYNAREP	B	TARLOC	B	TENND	O	THREEMILE	B	TISWORTH	C
SYRACUSE	B	TARPLEY	D	TENORIO	B	THROCK	C	TITUS	B/D
SYRENE	B/O	TARR	A	TENOT	C	THULEPAH	C	TITUSVILLE	C
SYRETT	C	TARRANT	O	TENPIN	O	THUMBERLANO	B	TIVDLI	A
TABECHEDING	C	TARRETE	O	TENRAG	B	THUNDERBIRD	O	TIVY	C
TABERNASH	B	TARRYALL	C	TENSAS	O	THURBER	D	TOA	B
TABLE MOUNTAIN	B	TASAYA	C	TENSED	C	THURLDNI	C	TOBICO	A/O
TABLER	O	TASCOSA	B	TENSLEEP	B	THURLOW	B	TDBIN	B
TABOR	O	TASSEL	O	TENSNOIR	B	THURMAN	A	TOBISH	C
TACAN	B	TASSELMAN	D	TENVDRRD	O	THURMONT	B	TOBLER	B
TACOMA	O	TASSO	B	TEOCULLI	B	TIAGDS	B	TDBOSA	O
TACOMA. ORAINED	C	TATE	B	TEPETE	O	TIAK	C	TDBY	B
TACONIC	C/O	TATIYEE	C	TEQUESTA	B/D	TIBAN	B	TOCAL	C
TACOOSH	B/D	TATLUM	O	TERAOA	B	TIBBITTS	B	TOCALOMA	C
TADLOCK	B	TATOUCHE	B	TERBIES	B	TIBS	C	TOCAN	C
TAFDYA	C	TATTON	O	TERENCE	B	TIBURONES	O	TOCCOA	B
TAFT	C	TATUM	C	TERESA	O	TICA	O	TDCK	C
TAFUNA	A	TAUNTON	C	TERINO	O	TICE	B	TOCOI	B/O
TAGGART	C	TAVARES	A	TERLCO	B	TICELL	O	TOOOLER	B
TAGLAKE	B	TAWAH	B	TERLINGUA	O	TICHNOR	O	TOOOSTAV	D
TAHKENITCH	B	TAWAS	A/O	TERMINAL	D	TICINO	C	TODOVILLE	B
TAHOMA	B	TAWCAW	C	TERMO	D	TICKASDN	B	TODDS	C
TAHOULA	O	TAYLDR	C	TERDMOTE	B	TIOINGS	B	TOEHEAD	C
TAHQUATS	B	TAYLOR CREEK	C	TEROUGE	D	TIOWELL	O	TOEJA	C
TAINTOR	C/D	TAYLORSFLAT	B	TERRA CEIA	B/D	TIERRA	O	TOEJA. NONGRAVELLY	B
TAJD	C	TAYLORSFLAT.	C	TERRA CEIA. TIDAL	D	TIETON	B	TDEW	C
TAKEUCHI	C	SALINE-ALKALI		TERRA CEIA.	B/D	TIFFANY	B/O	TOGNDNI	O
TAKILMA	B	TAYLORSVILLE	C	FREQUENTLY		TIFTON	B	TDGD	B
TAKOTNA	B	TAZLINA	A	FLOODED		TIGER CREEK	B	TOGUS	O
TALAG	O	TEAGULF	C	TERRAD	C	TIGERON	B	TOHONA	C
TALANTE	O	TEAKEAN	B	TERRETON	C	TIGIT	B	TDINE	B
TALAPUS	B	TEALSON	D	TERRIL	B	TIGIWON	B	TOISNOT	B/O
TALBDTT	C	TEALWHIT	D	TERRY	B	TIGON	O	TDISNOT. PONOEO	O
TALCOT	B/O	TEANAWAY	B	TERWILLIGER	O	TIGUA	O	TDIVOLA	A
TALIHINA	O	TEAPD	C	TERWILLIGER. STONY	C	TIJERAS	B	TOIYABE	C
TALKEETNA	C	TEASDALE	B	TESAJD	A	TIKI	O	TOKLAT	O
TALLA	C	TEBO	B	TESSFIVE	D	TILFER	B/O	TOKUL	C
TALLAC	B	TECHICK	B	TETHRICK	B	TILFORO	B	TOLANY	B
TALLAOEGA	C	TECO	D	TETON	C	TILLEOA	B	TDLBY	B
TALLAPDOSA	C	TECOLOTE	B	TETONIA	B	TILLICUM	B	TOLEOO	O
TALLEYVILLE	B	TECOPA	D	TETDNKA	C/O	TILLMAN	C	TOLEX	D
TALLS	B	TEDROW	B	TETONVIEW	O	TILLOU	C	TOLKE	B
TALLULA	B	TEEL	B	TETONVILLE	C	TILMA	C	TOLL	A
TALLY	B	TEELER	B	TETOTUM	C	TILSIT	C	TOLLGATE	B
TALMAGE	A	TEEMAT	B	TEVIS	B	TILTON	B	TOLLHOUSE	O
TALMO	A	TEETERS	C/O	TEW	C	TIMBALIER	O	TOLMAN	O
TALMOON	D	TEEWINOT	O	TEX	B	TIMBERG	C	TOLNA	B
TALOKA	D	TEFTON	C	TEXARK	O	TIMBERHEAD	B	TOLO	B
TALPA	O	TEGURO	D	TEXLINE	B	TIMBERVILLE	B	TOLONIER	B
TALQUIN	B/D	TEHACHAPI	C	TEXROY	B	TIMENTWA	B	TOLOVANA	B
TAMA	B	TEHAMA	C	TEZUMA	C	TIMHILL	D	TOLSONA	O
TAMAHA	D	TEHRAN	A	THACKER	C	TIMKEN	D	TOLSTOI	O
TAMALCO	D	TEIGEN	O	THACKERY	B	TIMMERMAN	A	TOLTEC	C
TAMALPAIS	C	TEJA	D	THADE[illegible]	C	TIMPAHUTE	O	TOLUCA	B
TAMANEEN	B	TEJABE	O	THAGE	C	TIMPANOGOS	B	TDLVAR	B
TAMBA	D	TEJANA	B	THATCHER	B	TIMPER	O	TOMAH	B
TAMELY	B	TEKENINK	B	THATUNA	C	TIMULA	B	TOMAHAWK	A
TAMFORO	O	TEKISON	C	THAYNE	B	TINA	C	TDMALES	O
TAMMANY CREEK	B	TEKLANIKA	A	THEBES	B	TINAJA	B	TOMAST	C
TAMP	B	TEKOA	C	THEBO	D	TINOAHAY	B	TOMBAR	C
TAMPICO	B	TELA	B	THEOALUND	C	TINE	A	TOME	B
TANAK	D	TELCHER	B	THENAS	C	TINEMAN	B	TOMEL	D
TANAMA	O	TELEFDNO	C	THEOOOR	D	TINEMAN. WET	C	TDMERA	D
TANANA	D	TELEPHDNE	D	THEDN	D	TINGEY	B	TOMICHI	A
TANBARK	O	TELESCOPE	A	THERESA	B	TINKER	C	TOMOKA	B/O
TANDY	C	TELFER	A	THERIOT	D	TINN	O	TOMOTLEY	B/O
TANEUM	B	TELFERNER	D	THERMOPOLIS	D	TINNIN	A	TOMSHERRY	C
TANEY	C	TELL	B	THESS	B	TINSLEY	A	TOMTY	D
TANGAIR	C	TELLER	B	THETFORO	A	TINTON	A	TDNASKET	B
TANNA	C	TELLICO	B	THETIS	B	TINYTDWN	B	TDNATA	D
TANNAHILL	B	TELLMAN	B	THIEFRIVER	B/D	TIOCANO	O	TONCANA	B
TANNER	C	TELLURA	C	THIEL	B	TIDGA	B	TONEY	D
TANOB	B	TELOS	C	THIOKOL	B	TIPPAH	C	TONGUE RIVER	C
TANSEM	B	TELSTAD	C	THISTLEDEW	B	TIPPECANOE	B	TONIO	B
TANTALUS	B	TEMBLOR	D	THOENY	O	TIPPER	C	TDNKA	C/O
TANTILE	C/D	TEMESCAL	D	THOMAS	D	TIPPERARY	A	TONKAWA	A
TANWAX	O	TEMO	C	THOMS	O	TIPPERARY. ALKALI	B	TONKEY	B/O
TANWAX. DRAINED	C	TEMPLE	C	THORNBURGH	B	TIPPERARY. DRY	A	TDNKIN	B
TANYARD	C	TEMPLETON	B	THORNOALE	O	TIPPO	C	TONKIN. MODERATELY	C
TAPCO	D	TEMVIK	B	THORNDIKE	C/D	TIPTON	B	WET	
TAPIA	B	TENABO	D	THORNOCK	D	TIPTONVILLE	B	TONKS	C
TAPICITOES	O	TENAHA	B	THORNTON	C	TIPTOP	B	TONOPAH	A
TAPPAN	B/D	TENAS	C	THOROUGHFARE	B	TIRO	C	TDNOR	C
TARA	B	TENCEE	O	THORP	C/D	TISBURY	B	TONOWEK	B
TARBORO	A	TENDOY	D	THOUT	C	TISCH	D	TONRA	B
TARGHEE	C	TENEX	B	THOW	B	TISOALE	C	TONSINA	B
TARKIO	O	TENINO	C	THRASH	B	TISHAR	C	TDNTI	C
TARKLIN	C	TENMILE	C	THREADGILL	B	TISDNIA	D	TDNUCO	D

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TABLE 7.1--HYDROLOGIC GROUPS OF THE SOILS OF THE UNITED STATES

TOOMES	D	TREATY	B/D	TUBAC	C	TWISSELMAN	C	UPSPRING	D
TOONE	C	TREBLE	B	TUCANNON	C	TWISSELMAN,	D	UPTMOR	C
TOP	C	TREBLOC	D	TUCKAHOE	B	SALINE-ALKALI,		UPTON	C
TOPEKI	D	TREBOR	C	TUCKER	C	WET		URACCA	B
TOPENAN	D	TREEKOR	D	TUCKERMAN	D	TWISSELMAN,	D	URBANA	C
TOPIA	D	TREGO	C	TUCSON	B	SALINE-ALKALI		URBO	D
TOPLIFF	B	TREHARNE	C	TUCUMCARI	C	TWOMILE	C/D	UREAL	D
TOPONCE	C	TRELK	B	TUFFIT	C	TWOTOP	D	URICH	C/D
TOPPENISH	D	TRELONA	D	TUGHILL	D	TYEE	D	URIPNES	C
TOPPENISH, DRAINED	C	TREMANT	B	TUJUNGA	A	TYGART	D	URLAND	C
TOPSEY	C	TREMBLES	B	TUKEY	C	TYGH	B	URNE	B
TOQUERVILLE	D	TREMONA	C	TUKWILA	D	TYLER	D	URNESS	B/D
TOQUI	D	TREMPE	A	TUKWILA, DRAINED	C	TYNDALL	C	URSA	C
TOQUOP	A	TREMPEALEAU	B	TULA	C	TYNER	A	URSINE	D
TORBOY	A	TRENARY	B	TULANA, DRAINED	C/D	TYRE	A/D	URTAH	C
TORCHLIGHT	C	TRENT	B	TULANA, NONFLOODED	C	TYRONE	C	USAL	B
TORDIA	D	TRENTON	D	TULARE	D	TYSON	C	USHAR	B
TOREX	A	TREON	D	TULARGO	B	UBANK	B	USK	C
TORHUNTA	C	TREP	B	TULAROSA	B	UBAR	D	UTABA	A
TORNEY	D	TRES HERMANOS	B	TULASE	B	UBIK	B	UTALINE	B
TORNILLO	B	TRESANO	B	TULIA	B	UBLY	B	UTE	C/D
TORNING	B	TRESED	C	TULIK	B	UCHEE	A	UTICA	B
TORODA	B	TRESTLE	B	TULLAHASSEE	C	UCOLO	D	UTLEY	B
TORONTO	C	TREVINO	D	TULLER	D	UCOPIA	B	UTSO	B
TORPEDO LAKE	D	TREVLAC	B	TULLOCK	C	UDAHO	B	UTUADO	B
TORREON	C	TREY	A	TULLY	C	UDEL	D	UVADA	D
TORRES	A	TRIANGLE	D	TULOSO	D	UDELOPE	D	UVALDE	B
TORRO	B	TRIBBEY	C	TUMAC	B	UDOLPHO	B/D	UVI	B
TORRY	B/D	TRICON	C	TUMALO	C	UFFENS	D	UWALA	B
TORSIDO	C	TRID	B	TUMWATER	C	UFFENS,	B	VABEM	D
TORTUGAS	D	TRIDELL	B	TUNBRIDGE	C	ELEVATION>5500		VABUS	C
TOSCA	B	TRIGGER	D	TUNICA	D	UGAK	D	VACHERIE	C
TOSTON	C	TRIGO	D	TUNIS	D	UHALDI	B	VADER	B
TOTAVI	A	TRIMAD	B	TUNKHANNOCK	A	UHL	B	VADNAIS	C
TOTELAKE	B	TRIMBLE	B	TUOMI	B	UHLAND	B	VADO	B
TOTEM	B	TRIMMER	C	TUPELO	D	UHLIG	B	VAEDA	D
TOTIER	C	TRINITY	D	TUPUKNUK	D	UHLORN	C	VAIDEN	D
TOTO	B/D	TRIO	D	TUQUE	B	UINTA	B	VAILTON	B
TOTTEN	C/D	TRIOMAS	B	TURBEVILLE	C	UKIAH	D	VAIVA	D
TOUCHET	B	TRIPIT	C	TURBOTVILLE	C	ULA	C	VALBY	C
TOUHEY	B	TRIPLEN	B	TURBYFILL	B	ULEN	B	VALCO	C
TOULON	B	TRIPOLI	B/D	TURK	C	ULIDA	D	VALCREST	C
TOURNQUIST	B	TRIPP	B	TURKEYSPRINGS	C	ULM	C	VALDEZ, CLAYEY	D
TOURS	B	TRISTAN	B	TURLEY	B	ULRIC	C	SUBSTRATUM	
TOUTLE	B	TRITON	D	TURLIN	B	ULRICHER	B	VALDEZ, DRAINED	C
TOUTLE, FLOODED	B	TRIX	B	TURLOCK	D	ULTRA	D	VALDOSTA	A
TOUTLE, PROTECTED	A	TROCKEN	B	TURNBULL	D	ULUPALAKUA	B	VALE	B
TOVAR	C	TROJAN	B	TURNER	B	ULY	B	VALENCIA	B
TOWAVE	B	TROMP	C	TURNERVILLE	B	ULYSSES	B	VALENT	A
TOWHEE	D	TRONSEN	B	TURNEY	B	UMA	A	VALENTINE	A
TOWNER	B	TROOK	B	TURRAH	D	UMAPINE	D	VALERA	C
TOWNLEY	C	TROOK, SALINE	C	TURRET	B	UMAPINE, DRAINED	C	VALHALLA	A
TOWNSEND	C	TROOK, GRAVELLY	B	TURRIA	C	UMATILLA	B	VALKARIA	B/D
TOWOSAHGY	B	SUBSTRATUM		TURSON	C	UMBARG	B	VALLAN	D
TOXAWAY	B/D	TROPAL	D	TUSAYAN	C	UMBERLAND	C	VALLE	B
TOY	D	TROPIC	B	TUSCAN	D	UMBERLAND, PONDED	D	VALLECITOS	D
TOYAH	B	TROSI	D	TUSCARAWAS	C	UMBERLAND, FLOODED	C	VALLEONO	B
TOYUSKA	B	TROSKY	B/D	TUSCAWILLA	D	UMIAT	D	VALLERS	C
TOZE	B	TROUGHS	D	TUSCOLA	B	UMIKOA	B	VALLEYCITY	D
TRABUCO	C	TROUP	A	TUSCOSSO	B	UMIL	D	VALMAR	C
TRACHUTE	B	TROUT CREEK	C	TUSCUMBIA	D	UMPA	B	VALMONT	C
TRACK	D	TROUT RIVER	A	TUSEL	B	UMPCOOS	C	VALMY	B
TRACK, DRAINED	C	TROUTDALE	B	TUSK	B	UMPUMP	B	VALNOR	C
TRACOSA	D	TROUTER	C	TUSKAHOMA	D	UNA	D	VALOIS	B
TRACY	B	TROUTVILLE	B	TUSKEEGO	C/D	UNADILLA	B	VALSETZ	C
TRADEDOLLAR	B	TROXEL	B	TUSLER	B	UNAKA	B	VALTO	D
TRAER	B/D	TRUCE	C	TUSQUITEE	B	UNAKWIK	D	VALTON	C
TRAG	B	TRUCHOT	C	TUSSY	D	UNAWEEP	B	VALVERDE	B
TRAG, DRY	B	TRUCKEE	C	TUSTIN	B	UNCAS	D	VAMER	D
TRAG, COOL	C	TRUCKEE, SALINE	C	TUSTUMENA	B	UNCOMPAHGRE	B/D	VAMONT	D
TRAHAM	C	TRUCKEE, DRAINED	B	TUTE	B	UNDERHILL	B	VAMP	C
TRAIL	A	TRUCKEE, SANDY	C	TUTHILL	B	UNDERWOOD	B	VAN DUSEN	B
TRAMPAS	C	SUBSTRATUM		TUTNI	B	UNDUSK	B	VAN HORN	B
TRAMWAY	B	TRUCKEE, GRAVELLY	C	TUTWILER	B	UNGERS	B	VAN NOSTERN	C
TRANQUILAR	C	SUBSTRATUM		TUWEEP	B	UNICOI	B	VAN WAGONER	D
TRANSYLVANIA	B	TRUCKTON	B	TWEBA	B/D	UNION	C	VANAJO	D
TRAPPER	B	TRUDAU	B	TWEBA,	B/D	UNIONTOWN	B	VANANDA	D
TRAPPIST	C	TRUDE	B	SALINE-ALKALI		UNIONVILLE	B	VANCE	C
TRAPPS	B	TRUEFISSURE	A	TWEBA, MODERATELY	B	UNISON	B	VANDA	D
TRASK	C	TRUESDALE	C	WET		UNIUS	D	VANDALIA	D
TRAVELERS	D	TRULON	C	TWEEDY	C	UNLIC	B	VANDAMME	C
TRAVER	B	TRUMAN	B	TWICK	D	UNSEL	B	VANDAMORE	B
TRAVERTINE	C	TRUMBULL	D	TWIG	D	UNSON	B	VANDERGRIFT	C
TRAVESSILLA	D	TRUMP	D	TWILIGHT	B	UPDIKE	D	VANDERHOFF	C
TRAVIS	C	TRUNK	C	TWIN CREEK	B	UPSATA	A	VANDERLIP	A
TRAWICK	B	TRYON	D	TWINING	C	UPSHUR	D	VANEPPS	C
TRAY	C	TSCHICOMA	B	TWINLAKE	C	UPSON	B	VANET	D
TREADWAY	D	TUB	C	TWINSI	C	UPSON, STONY	C	VANG	B

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VANGUARD	D	VERNONIA	B	VIRTUE	C	WAINEE	B	WARM SPRINGS, VERY	D
VANMETER	C	VERO	B/D	VISTA	B	WAINOLA	B	POORLY DRAINED	
VANNI	B	VERONA, WET	C	VITALE	C	WAIPAHU	C	WARMAN	B/D
VANNOY	C	VERONA, DRAINED	B	VIUDA	D	WAISKA	B	WARMAN, GRAVELLY	A/D
VANOCKER	B	VERONA, FLOODED	B	VIVES	B	WAITS	B	SUBSOIL	
VANOSS	B	VERONA, CLAY	C	VIVI	B	WAKE	D	WARNEKE	D
VANPETTEN	B	SUBSTRATUM		VLASATY	C	WAKEEN	B	WARNERS	C/D
VANSON	B	VERSHIRE	C	VOATS	B	WAKEFIELD	B	WARRENTON	D
VANSTEL	B	VERSON	C	VOCA	C	WAKELAND	C	WARSAW	B
VANWYPER	C	VERTEL	D	VODERMAIER	B	WAKITA	1	WARSING	B
VANZANDT	C	VERTREES	B	VOIGHT	B	WAKONDA	C	WARWICK	A
VAQUERO	D	VES	B	VOLADORA	B	WAKULLA	A	WASA	D
VARCO	D	VESEY	B	VOLASH	B	WALCOTT	B	WASCO	B
VARELUM	B	VESPER	D	VOLBORG	D	WALDBILLIG	B	WASDA	B/D
VARELUM, CLAY LOAM	C	VESSER	C	VOLCO	D	WALDECK	C	WASEPI	B
SUBSTRATUM		VESSILLA	D	VOLENTE	C	WALDEN	D	WASHBURN	D
VARICK	D	VESTA	B	VOLINIA	B	WALDO	D	WASHINGTON	B
VARINA	C	VESTABURG	A/D	VOLKMAR	B	WALDORF	C/D	WASHINGTON, WET	C
VARNA	C	VESTON	D	VOLNEY	B	WALDPORT	A	SUBSTRATUM	
VARNEY	B	VETA	B	VOLPERIE	C	WALDRON	D	WASHINGTON, STONY	B
VARRO	B	VETAL	B	VOLTA	D	WALDROUP	D	WASHINGTON,	B
VARYSBURG	B	VETEADO	C	VOLTAIRE	D	WALES	B	GRAVELLY	
VASA	B	VEYO	D	VOLTAIRE,	D	WALES, WARM	B	WASHOE	B
VASHTI	C	VIA	B	SALINE-ALKALI		WALES, OVERBLOWN	C	WASHOUGAL	B
VASQUEZ	B	VIAN	B	VOLTAIRE, WET	D	WALFORD	B/D	WASHTENAW	C/D
VASSALBORO	D	VIBLE	A	VOLTAIRE, DRAINED	C	WALHALLA	B	WASILLA	C
VASSAR	B	VIBO	B	VOLTAIRE, FLOODED	D	WALKE	C	WASIOJA	B
VASTINE	C	VIBORAS	D	VOLTAIRE, GRAVELLY	D	WALKNOLLS	D	WASKISH	D
VAUCLUSE	C	VIBORG	B	SUBSTRATUM		WALKON	D	WASKOM	C
VAUGHAN	D	VICEE	B	VOLUSIA	C	WALL	B	WASPO	D
VAUGHNSVILLE	C	VICK	C	VONA	B	WALLA WALLA	B	WASSAIC	B
VAY	B	VICKERY	C	VOORHIES	C	WALLACE	B	WATAB	C
VAYAS	D	VICKING	B	VORE	B	WALLEN	B	WATAMA	C
VEAL	B	VICKING, HIGH	B	VOSBURG	B	WALLER	B/D	WATAUGA	B
VEATCH	B	RAINFALL		VOSS	B	WALLINGTON	C	WATCHABOB	C
VEATCH, STONY	C	VICKING, DRY	D	VOSSET	A	WALLKILL	C/D	WATCHAUG	B
VEAZIE	B	VICKSBURG	B	VULCAN	C	WALLROCK	C	WATCHUNG	D
VEBAR	B	VICTINE	D	VYLACH	D	WALLSBURG	D	WATERBURY	D
VECONT	D	VICTOR	B	WAAS	B	WALLSON	B	WATERCANYON	B
VEEDUM	D	VICTORIA	D	WABASH	D	WALLUSKI	B	WATEREE	B
VEET	B	VICTORVILLE	B	WABASHA	D	WALNETT	C	WATERMAN	D
VEGA	C	VICTORY	B	WABASSO	B/D	WALONG	B	WATERTOWN	A
VEGA ALTA	B	VICU	C	WABBASEKA	D	WALPOLE	C	WATKINS	B
VEGA BAJA	C	VIDA	B	WABEK	A	WALREES	B	WATKINS RIDGE	B
VEKOL	D	VIDAURI	D	WABEN	B	WALSH	B	WATO	B
VEKOL, COOL	C	VIDRINE	D	WABUSKA	C	WALSTEAD	B	WATONGA	D
VELASCO	D	VIEJA	D	WACA	B	WALTERS	B	WATROUS	B
VELOA	B	VIENNA	B	WACAHOOTA	D	WALTI	C	WATSEKA	B
VELDKAMP	B	VIEQUES	B	WACOTA	B	WALUM	B	WATSON	C
VELMA	B	VIGAR	C	WACOUSTA	B/D	WALVAN	B	WATSONIA	D
VELOW	B	VIGIA	D	WADAMS	B	WALVILLE	B	WATSONVILLE	D
VELVA	B	VIGO	D	WADDOUPS	B	WAMBA	D	WATT	D
VENA	C	VIGUS	B	WADELL	B	WAMBA, DRAINED	B	WATTON	C
VENABLE	B/D	VIKING	D	WADENA	B	WAMIC	B	WATUSI	C
VENABLE, STONY	C	VIL	D	WADENILL	B	WAMPOO	D	WAUBAY	B
VENANGO	C	VILAS	A	WADER	C	WAMPSVILLE	B	WAUBEEK	B
VENATOR	C	VILLA	B	WADESPRING	B	WANBLEE	D	WAUBERG	D
VENETA	D	VILLA GROVE	B	WADMALAW	D	WANDO	A	WAUBONSIE	B
VENEZIA	D	VILLEGREEN	C	WADSWORTH	C	WANETTA	B	WAUCHULA	B/D
VENICE	C	VILLY	B	WAGES	B	WANILLA	C	WAUCHULA,	D
VENLO	D	VILLY,	D	WAGNER	D	WANN	B	DEPRESSIONAL	
VENTRIS	D	ELEVATION>5500		WAGONBOX	D	WANOGA	B	WAUCOBA	D
VENTURE	D	VILLY, DRAINED	B	WAGONTIRE	D	WANSER	D	WAUCOMA	B
VENUM	B	VILOT	C	WAGRAM	A	WANSER, DRAINED	B	WAUCONDA	B
VENUS	B	VINVILLE	D	WAHA	C	WAPAL	A	WAUKEE	B
VERBOORT	D	VINA	B	WAHATOYA	C	WAPATO	D	WAUKEGAN	B
VERDE	C	VINCENNES	C/D	WAHEE	D	WAPELLO	B	WAUKENA	D
VERDEL	D	VINCENT	C	WAHIAWA	B	WAPI	D	WAUKON	B
VERDICO	D	VINEGARROON	C	WAHIKULI	C	WAPINITIA	B	WAULD	B
VERDIGRIS	B	VINEYARD	C	WAHKEENA	B	WAPPING	B	WAUMBEK	B
VERDUN	D	VINGO	B	WAHLUKE	B	WAPPINGER	B	WAUNA	D
VERGAS	C	VINING	C	WAHOO	D	WAPSHILLA	B	WAUNA, PROTECTED	C
VERGENNES	D	VININI	D	WAHPETON	C	WAPSIE	B	WAUPACA	B/D
VERHALEN	D	VINITA	C	WAHREKDAM	C	WAPTUS	C	WAUPECAN	B
VERICK	C	VINLAND	D	WAHTIGUP	B	WARBA	B	WAURIKA	D
VERLOT	C	VINSAD	C	WAHTUM	D	WARDBORO	A	WAUSEON	B/D
VERMEJO	D	VINSON	B	WAIAHA	D	WARDELL	C	WAUTOMA	B/D
VERMILAC	C	VINT	B	WAIAKOA	C	WARDEN	B	WAVELAND	B/D
VERMILLON	C	VINTAS	A	WAIALEALE	D	WARDENOT	A	WAVERLY	B/D
VERMISA	D	VINTON	B	WAIALUA	B	WARDWELL	C	WAWASEE	B
VERNAL	B	VIPONT	C	WAIAWA	D	WARE	B	WAWINA	A
VERNALIS	B	VIRATON	C	WAIHUNA	D	WAREHAM	C	WAX	C
VERNALIS,	B	VIRDEN	B/D	WAIKALOA	B	WARM SPRINGS	C	WAYBE	D
SALINE-ALKALI		VIRGELLE	C	WAIKANE	B	WARM SPRINGS,	C	WAYCUP	B
VERNALIS, WET	C	VIRGIL	B	WAIKAPU	B	ALKALI		WAYDEN	D
VERNIA	A	VIRGIN PEAK	D	WAIKOMO	D	WARM SPRINGS, WET	D	WAYLAND	C/D
VERNIGOR	C	VIRGIN RIVER	C	WAILUKU	B	WARM SPRINGS, CLAY	C	WAYMOR	B
VERNON	D	VIRKULA	C	WAIMEA	B	SUBSTRATUM		WAYNECO	D

NOTES: TWO HYDROLOGIC SOIL GROUPS SUCH AS B/C INDICATES THE DRAINED/UNDRAINED SITUATION.
MODIFIERS SHOWN, E.G., BEDROCK SUBSTRATUM, REFER TO A SPECIFIC SOIL SERIES PHASE FOUND IN SOIL MAP LEGEND.

TABLE 7.1--HYDROLOGIC GROUPS OF THE SOILS OF THE UNITED STATES

WAYNESBORO	B	WESTBURY	C	WHOBREY	D	WINDY	B	WOLLARD	C
WAYNETOWN	C	WESTBUTTE	C	WHOLAN	B	WINEG	B	WOLLENT	D
WEA	B	WESTCAMP	C	WIBAUX	B	WINEMA	C	WOLDT	B
WEASH	C	WESTCREEK	B	WICHITA	C	WINETTI	B	WOLVERINE	A
WEATHERFORD	B	WESTERVILLE	B	WICHUP	D	WINEVADA	C	WOO	B
WEAVER	C	WESTHAVEN	C	WICKAHONEY	D	WINFALL	B	WOO, OVERWASH	C
WEBB	C	WESTLAKE	D	WICKENBURG	D	WINFIELD	B	WOO, WET	C
WEBBRIDGE	B	WESTLAND	B/D	WICKERSHAM	B	WING	D	WOO, GRAVELLY	B
WEBBTOWN	C	WESTMORE	C	WICKETT	C	WINGATE	B	SUBSTRATUM	
WEBER	B	WESTMORELAND	B	WICKHAM	B	WINGER	B/D	WOOD RIVER	D
WEBILE	C	WESTON	D	WICKIUP	C	WINGINAW	D	WOODBECK	B
WEBSTER	B/D	WESTOVER	B	WICKSBURG	B	WINGVILLE	D	WOODBINE	B
WEDEKIND	D	WESTPHALIA	B	WIDEMAN	A	WINIFRED	C	WOODBRIDGE	C
WEDERTZ	B	WESTPLAIN	C	WIDEN	C	WINK	B	WOODBURN	C
WEDGE	A	WESTPORT	B	WIDTSOE	B	WINKEL	D	WOODBURY	D
WEDLAR	C	WESTVACO	C	WIEHL	C	WINKLEMAN	C	WOODCOCK	B
WEDOWEE	B	WESTVIEW	B	WIELAND	C	WINKLEMAN, SALINE	C	WOODFORD	D
WEED	B	WESTVILLE	B	WIERGATE	D	WINKLEMAN, WET	D	WOODGULCH	A
WEEDING	D	WESTWEGO	D	WIGGLETON	B	WINKLER	B	WOODHALL	C
WEEDMARK	B	WESWOOD	B	WIGTON	A	WINLER	D	WOODHURST	C
WEEKIWACHEE	D	WETA	D	WILAHA	B	WINLO	D	WOODIN	B
WEEKS	C	WETHERSFIELD	C	WILBANKS	D	WINN	C	WOODINGTON	B/D
WEEKSVILLE	B/D	WETHEY	C	WILBRAHAM	C	WINNEBAGO	B	WOODINVILLE	D
WEENA	D	WETHEY, DRAINED	A	WILBUR	B	WINNECONNE	C	WOODINVILLE,	C
WEEPAH	C	WETMORE	D	WILBURTON	B	WINNECOOK	C	DRAINED	
WEESATCHE	B	WETSAW	C	WILCO	C	WINNEMUCCA	B	WOODLAWN	B
WEGA	B	WETTERHORN	C	WILCOX	D	WINNESHIEK	B	WOODLY	B
WEHADKEE	D	WETZEL	D	WILCOXSON	D	WINNETT	D	WOODLYN	D
WEIGANG	C	WEWELA	B	WILDALE	C	WINNSBORO	D	WOODLYN	D
WEIGLE	D	WEWOKA	C	WILDCAT	D	WINONA	D	WOODLYN, DRAINED	C
WEIKERT	C/D	WEYMOUTH	B	WILDERNESS	C	WINOOSKI	B	WOODLYN, DRAINED	D
WEIMER	D	WHAKANA	B	WILDHORSE	A	WINOPEE	B	WOODMANSIE	B
WEINBACH	C	WHALAN	B	WILDORS	C	WINRIDGE	C	WOODMERE	B
WEINGART	D	WHALEY	D	WILDWOOD	D	WINSHIP	C	WOODMONT	C
WEINGARTEN	C	WHARTON	C	WILE	C	WINSPECT	B	WOODPASS	B
WEIR	D	WHATCOM	B	WILEY	B	WINSTON	B	WOODROCK	C
WEIRMAN	A	WHATELY	D	WILHOIT	B	WINTERFIELD	A/D	WOODROW	B
WEIRMAN, WET	D	WHEATLEY	A/D	WILKES	C	WINTERHAVEN	B	WOODROW,	C
WEIRMAN,	A	WHEATRIDGE	B	WILKESON	B	WINTERIDGE	B	SALINE-ALKALI	
NONFLOODED		WHEATVILLE	B	WILKINS	D	WINTERS	C	WOODS CROSS	D
WEISBURG	C	WHEELER	B	WILL	B/D	WINTERSBURG	C	WOODSEYE	D
WEISER	B	WHEELERVILLE	B	WILLABY	C	WINTERSET	C	WOODSFIELD	C
WEISHAUPT	D	WHEELING	B	WILLACY	B	WINTHROP	A	WOODSIDE	A
WEITCHPEC	C	WHEELON	D	WILLAKENZIE	C	WINTON	C	WOODSON	D
WELAKA	A	WHETROCK	C	WILLAMAR	B	WINTONER	B	WOODSTOCK	C/D
WELBY	B	WHETSTONE	C	WILLAMETTE	B	WINU	C	WOODSTOWN	C
WELCH	D	WHICHMAN	B	WILLAMETTE, WET	C	WIOTA	B	WOODTELL	D
WELCH, DRAINED	B	WHIDBEY	C	WILLAMETTE,	B	WIRT	B	WOODVILLE	D
WELCHLAND	B	WHIPPANY	C	GRAVELLY		WISCOW	D	WOODWARD	B
WELD	C	WHIPSTOCK	C	SUBSTRATUM		WISE	C	WOOLPER	C
WELDA	C	WHIRLO	B	WILLANCH	C	WISEMAN	A	WOOLSEY	B
WELLER	C	WHISPERING	C	WILLAPA	B	WISENOR	D	WOOLSTALF	B
WELLINGTON	D	WHISTLE	B	WILLARD	B	WISERLAKE	D	WOOLSTED	B
WELLMAN	B	WHIT	B	WILLETTE	A/D	WISHARD	B/D	WOONSOCKET	B
WELLS	B	WHITAKER	C	WILLHILL	C	WISHBONE	B	WOOSLEY	C
WELLSBORO	C	WHITE HOUSE	C	WILLIAMS	B	WISHEYLU	C	WOOSTER	C
WELLSED	C	WHITE STORE	D	WILLIAMSBURG	B	WISHKAH	D	WORCESTER	C
WELLSTON	B	WHITE SWAN	C	WILLIAMSON	C	WISHKAH, DRAINED	C	WORF	D
WELLSVILLE	B	WHITECAP	D	WILLIAMSTOWN	C	WISKAN	C	WORFKA	D
WELLTON	B	WHITECLOUD	B	WILLIAMSVILLE	C	WISNER	B/D	WORFMAN	D
WELOY	C	WHITECOW	B	WILLIMAN	B/D	WISTER	C	WORK	C
WELRING	D	WHITEFISH	B	WILLIS	C	WITBECK	B/D	WORL	B
WELTER	D	WHITEFORD	B	WILLISTON	C	WITEFELS	B	WORLAND	C
WEMPLE	B	WHITEHALL	B	WILLOW CREEK	B	WITHAM	D	WORLEY	C
WENAS	D	WHITEHILLS	C	WILLOWDALE	B	WITHEE	C	WORMSER	D
WENAS, DRAINED	C	WHITEHORN	D	WILLOWMAN	B	WITHERBEE	A/D	WORNOCK	B
WENATCHEE	C	WHITEHORSE	B	WILLOWS	D	WITHERELL	D	WORSHAM	D
WENDANE	C	WHITEKNOB	B	WILLWOOD	A	WITHERS	C	WORTH	C
WENDANE, DRAINED	B	WHITELAKE	B	WILMER	C	WITT	B	WORTHEN	B
WENDOVER	D	WHITEMAN	D	WILMONTON	B	WITTEN	D	WORTHING	D
WENDTE	D	WHITEPEAK	D	WILPOINT	D	WITTENBERG	B	WORTMAN	D
WENONA	C	WHITEROCK	D	WILSHIRE	A	WITZEL	D	WOVOKA	D
WENTWORTH	B	WHITESBORO	C	WILSON	D	WIX	C	WRANGO	A
WENZEL	C	WHITESBURG	C	WILSONVILLE	C	WIXOM	B	WREDAH	B
WEOGUFKA	C	WHITESON	D	WILSOR	B	WOCKLEY	C	WRENCOE	D
WERLOG,	C	WHITEWATER	D	WILTON	B	WODEN	B	WRENMAN	C
SALINE-ALKALI		WHITEWOLF	A	WINADA	C	WODSKOW	B	WRENTHAM	C
WERLOG, FLOODED	C	WHITEWOOD	C/D	WINBERRY	C	WOHLY	B	WRIGHT	C
WERLOG, NONFLOODED	B	WHITEWRIGHT	C	WINCHESTER	A	WOLCO	C	WRIGHTMAN	C
WERLOG, COOL	C	WHITLEY	B	WINCHUCK	C	WOLCOTT	B/D	WRIGHTSBORO	C
WERNER	D	WHITLOCK	B	WIND RIVER	B	WOLDALE	D	WRIGHTSVILLE	D
WERNOCK	B	WHITMAN	D	WINDER	B/D	WOLDALE, DRAINED	C	WRIGHTWOOD	B
WESCONNETT	D	WHITNEY	C	WINDHAM	B	WOLF	B	WUKOKI	B
WESKA	D	WHITORE	B	WINDMILL	B	WOLF POINT	C	WUKSI	A
WESLEY	B	WHITSOL	B	WINDSOR	A	WOLFESON	C	WULFERT	D
WESO	B	WHITSON	D	WINDTHORST	C	WOLFESON, WET	D	WUNJEY	B
WESSEL	C	WHITTIER	B	WINDWHISTLE	C	WOLFPEN	A	WUPATKI	D
WESTBROOK	D	WHITWELL	C	WINDWHISTLE, WARM	B	WOLFTEVER	C	WURNO	C

NOTES: TWO HYDROLOGIC SOIL GROUPS SUCH AS B/C INDICATES THE DRAINED/UNDRAINED SITUATION.
MODIFIERS SHOWN, E.G., BEDROCK SUBSTRATUM, REFER TO A SPECIFIC SOIL SERIES PHASE FOUND IN SOIL MAP LEGEND.

TABLE 7.1--HYDROLOGIC GROUPS OF THE SOILS OF THE UNITED STATES

WURSTEN	B	YEOMAN	C	ZAZA	D
WURTSBORO	C	YEOPIM	B	ZEALE	B
WYALUSING	D	YERINGTON	A	ZEB	B
WYANDOTTE	D	YERMO	B	ZECANYON	C
WYARD	B	YESO	D	ZEEBAR	B
WYARNO	B	YESUM	B	ZEEKA	C
WYATT	C	YETTEM	B	ZEESIX	C
WYE	B	YETULL	A	ZEGRO	C
WYEAST	C	YIPOR	B	ZEIBRIGHT	B
WYETH	B	YOBE	C	ZELL	B
WYEVILLE	C	YOCHUM	C	ZEN	C
WYKOFF	B	YOCKEY	C	ZENDA	C
WYMAN	B	YODER	B	ZENI	C
WYMORE	D	YODY	C	ZENIA	C
WYNDMERE	B	YOHURT	D	ZENIFF	B
WYNN	B	YOKAYO	D	ZENITH	B
WYNNVILLE	C	YOKOHL	D	ZENKER	B
WYNONA	C	YOLLABOLLY	D	ZENOD	B
WYNOOSE	D	YOLO	B	ZENOR	B
WYOCENA	B	YOLOGO	D	ZEONA	A
WYOMING	A	YOMBA	B	ZEORELY	B
WYRENE	B	YOMONT	B	ZEPHAN	C
WYSOCKING	C/D	YONCALLA	D	ZEPHYR	D
XAVIER	B	YONGES	D	ZERKER	B
XENIA	B	YONNA	D	ZIEGENFUSS	D
XERTA	D	YORBA	D	ZIEGLER	C
XINE	C	YORK	C	ZIGWEID	B
XMAN	D	YORKTOWN	D	ZILABOY	D
YACOLT	B	YORKTREE	C	ZILLAH	D
YAGO	C	YORKVILLE	D	ZILLAH, DRAINED	C
YAHARA	C	YOST	D	ZILLION	B
YAHNE	C	YOST, DRAINED	C	ZILLMAN	B
YAHOLA	B	YOUD	D	ZIMMERMAN	A
YAINAX	B	YOUGA	B	ZINEB	B
YAKI	D	YOUJAY	D	ZING	C
YAKIMA	B	YOUMAN	C	ZINZER	B
YAKUS	D	YOUNGSTON	B	ZINZER, MODERATELY	B
YAKUTAT	A	YOUNGSTON,	B	SLOW PERM	
YALELAKE	B	ELEVATION>5200		ZINZER, SALINE	C
YALESVILLE	C	YOUNGSTON,	B	ZINZER, HIGH	B
YALLANI	B	MODERATELY WET		RAINFALL	
YALMER	B	YOUNGSTON, WET	D	ZION	C
YAMAC	B	YOUNGSTON, DRY	B	ZIPP	D
YAMHILL	C	YOUNGSTON,	B	ZIRAM	C
YAMSAY	C/D	OCCASIONALLY		ZITA	B
YANA	B	FLOODED		ZITTAU	C
YANCY	D	YOURAME	B	ZOAR	C
YANKEE	D	YOUTLKUE	D	ZOATE	D
YANKTON	B	YOVIMPA	D	ZOE	D
YANUSH	B	YPSI	C	ZOESTA	C
YAP	B	YRIBARREN	D	ZOHNER	D
YAPOAH	B	YSIDORA	C	ZOLA	C
YAQUINA	D	YTURBIDE	A	ZOLFO	C
YARCO	D	YTURRIA	A	ZOLTAY	C
YARDLEY	C	YUBA	D	ZOOK	C/D
YARTS	B	YUKO	D	ZORRA	D
YATAHONEY	C	YUKON	D	ZORRAVISTA	A
YATES	D	YULEE	D	ZUBER	C
YAUCO	C	YUNES	D	ZUFELT	C
YAUHANNAH	B	YUNQUE	C	ZUKAN	D
YAWDIM	D	YUTRUE	D	ZUMAN	C/D
YAWHEE	B	YUVAS	D	ZUMBRO	A
YAWKEY	C	ZAAR	D	ZUMWALT	C
YAXON	B	ZABA	B	ZUNDELL	C
YEARY	C	ZACA	D	ZUNHALL	C
YEATES HOLLOW	B	ZACHARIAS	B	ZUNI	D
YEATES HOLLOW,	C	ZACHARY	C	ZURICH	B
LOAMY SUBSTRATUM,		ZADOG	A/D	ZWIEFEL	C
STONY		ZAFRA	B	ZWINGLE	D
YEATES HOLLOW,	C	ZAGG	C	ZYGORE	B
LOAMY SUBSTRATUM		ZAHILL	B	ZYME	D
YEATES HOLLOW,	C	ZAHL	B	ZYMER	B
STONY		ZAIDY	C	ZYNBAR	B
YEATES HOLLOW,	C	ZAKME	D	ZYZYL	B
NONSTONY		ZALCO	A	ZYZZI	D
YECROSS	A	ZALOA	D		
YEDLICK	B	ZALLA	A		
YEGEN	B	ZAMORA	B		
YELJACK	B	ZAMSCAN	B		
YELLOWBAY	B	ZANE	B		
YELLOWHOUND	B	ZANEIS	B		
YELLOWROCK	A	ZANESVILLE	C		
YELLOWSTONE	D	ZANGO	D		
YELM	C	ZAPATA	C		
YEMASSEE	C	ZAU	C		
YENCE	C	ZAVALA	B		
YENLO	B	ZAVCO	C		
YENRAB	A	ZAYANTE	A		

NOTES: TWO HYDROLOGIC SOIL GROUPS SUCH AS B/C INDICATES THE DRAINED/UNDRAINED SITUATION.
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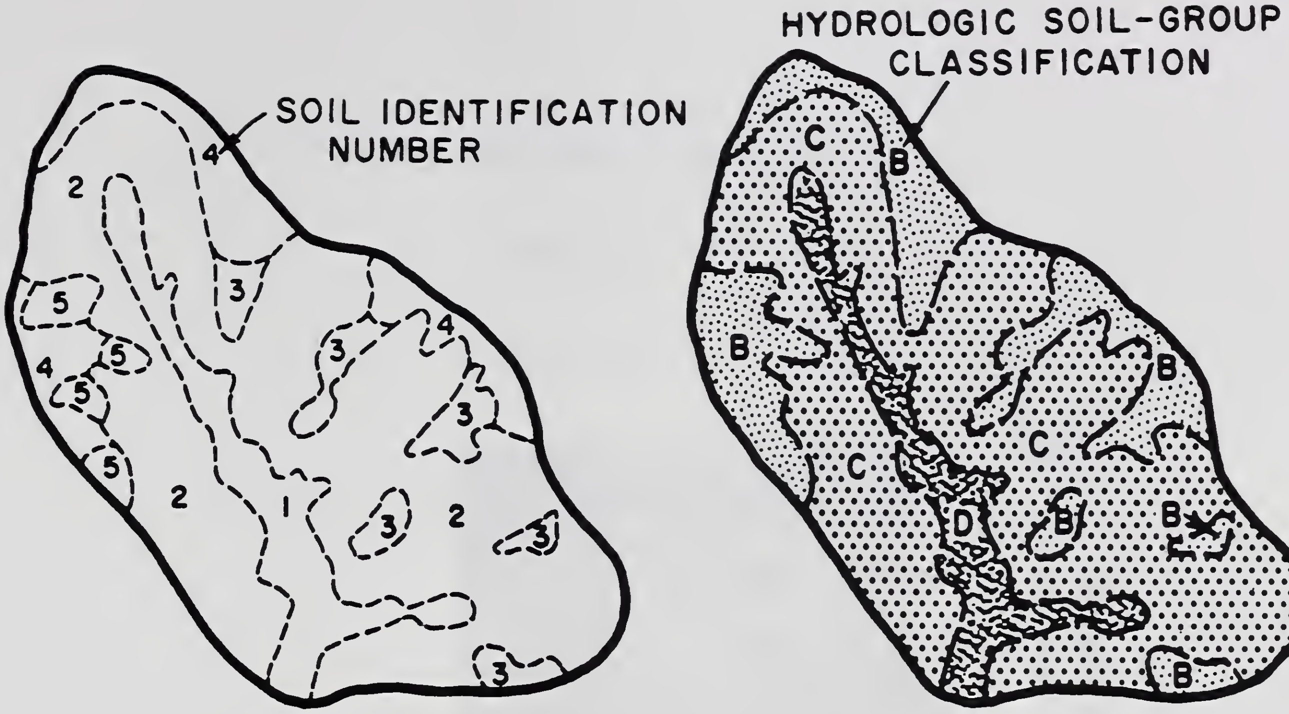
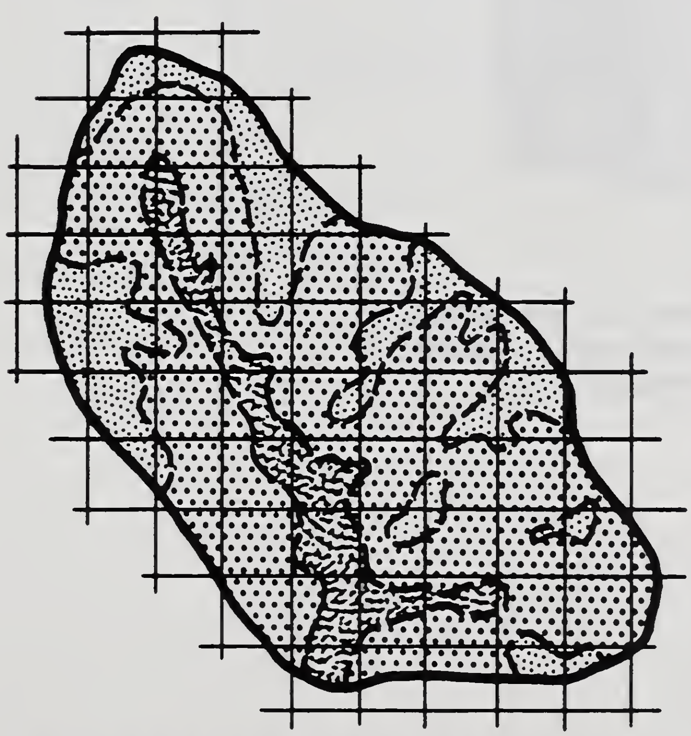


(a) DETAILED SOILS MAP (b) HYDROLOGIC SOIL GROUP MAP



(c) GRID ON SOIL GROUP MAP

SOIL GROUP	NUMBER GRID INTERSECTIONS	PERCENT
B	12	23*
C	32	63
D	7	14
TOTAL	51	100

*PERCENT FOR B:

$$(100)\frac{12}{51} = \underline{\underline{23}}$$

(d) COMPUTATIONS

Figure 7.1.--Steps in determining percentages of soil groups.

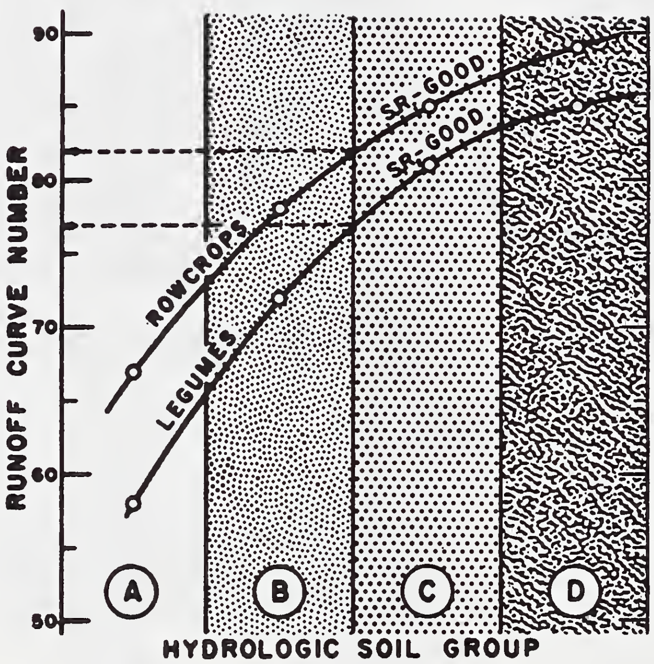


Figure 7.2.--Type of plotting used in estimating runoff curve-numbers for soil subgroups. Dashed lines show results for example 7.1.

NATIONAL ENGINEERING HANDBOOK

SECTION 4

HYDROLOGY

CHAPTER 8. LAND USE AND TREATMENT CLASSES

by

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SCS NATIONAL ENGINEERING HANDBOOK

SECTION 4

HYDROLOGY

CHAPTER 8--LAND USE AND TREATMENT CLASSES

CONTENTS

CHAPTER 8. LAND USE AND TREATMENT CLASSES

The land use and treatment classes ordinarily evaluated in watershed studies are briefly described. These classes are used in determining hydrologic soil-cover complexes (chap. 9), which are used in a method for estimating runoff from rainfall (chap. 10).

Classification of Land Use and Treatment

In the SCS method of runoff estimation the effects of the surface conditions of a watershed are evaluated by means of land use and treatment classes. Land use is the watershed cover and it includes every kind of vegetation, litter and mulch, and fallow (bare soil, to which the classification of chapter 7 also applies) as well as nonagricultural uses such as water surfaces (lakes, swamps, etc.) and impervious surfaces (roads, roofs, etc.). Land treatment applies mainly to agricultural land uses and it includes mechanical practices such as contouring or terracing and management practices such as grazing control or rotation of crops. The classes consist of use and treatment combinations actually to be found on watersheds.

Land use and treatment classes are readily obtained either by observation or by measurement of plant and litter density and extent on sample areas.

CLASSES

The land use and treatment classes discussed here are listed in table 9.1, which also shows the runoff curve numbers (CN) for hydrologic soil-cover complexes in which the classes are used. Agricultural terms not defined here are defined in the glossary (chap. 22).

Cultivated Land

Fallow listed in table 9.1 is the agricultural land use and

treatment with the highest potential for runoff because the land is kept as bare as possible to conserve moisture for use by a succeeding crop. The loss due to runoff is offset by the gain due to reduced transpiration. Other kinds of fallow, such as stubble-mulch, are not listed but they can be evaluated by comparing their field condition with those for classes that are listed.

Row crop is any field crop (maize, sorghum, soybeans, sugar beets, tomatoes, tulips) planted in rows far enough apart that most of the soil surface is exposed to rainfall impact throughout the growing season. At planting time it is equivalent to fallow and may be so again after harvest. In most evaluations average seasonal condition is assumed but special conditions can be evaluated as shown in chapter 10. Row crops are planted either in straight rows or on the contour and they are in either a poor or good rotation. These land treatments are discussed later in this chapter.

Small grain (wheat, oats, barley, flax, etc.) is planted in rows close enough that the soil surface is not exposed except during planting and shortly thereafter. Land treatments are those used with row crops.

Close-seeded legumes or rotation meadow (alfalfa, sweetclover, timothy, etc. and combinations) are either planted in close rows or broadcast. This cover may be allowed to remain for more than a year so that year-round protection is given to the soil. The land treatments used with row crops are also used with this cover, except for row treatments if the seed is broadcast.

Rotations are planned sequences of crops, and their purpose is to maintain soil fertility or reduce erosion or provide an annual supply of a particular crop. Hydrologically, rotations range from "poor" to "good" in proportion to the amount of dense vegetation in the rotation, and they are evaluated in terms of hydrologic effects. Poor rotations are generally one-crop land uses such as continuous corn (maize) or continuous wheat or combinations of row crops, small grains, and fallow. Good rotations generally contain alfalfa or other close-seeded legume or grass to improve tilth and increase infiltration. Their hydrologic effects may carry over into succeeding years after the crop is removed though normally the effects are minor after the second year. The carry-over effect is not considered in table 9.1.

Straight-row fields are those farmed in straight rows either up and down the hill or across the slope. Where land slopes are less than about 2 percent, farming across the slope in straight rows is equivalent to contouring and should be so considered when using table 9.1. Contoured fields are those farmed as nearly as possible on the contour. The hydrologic effect of contouring is due to the surface storage provided by the furrows because the storage prolongs the time during which infiltration can take place. The magnitude of storage depends not only on the dimensions of the furrows but also on the land slope, crop, and manner of planting and cultivation. Planting small

grains or legumes on the contour makes small furrows that disappear because of climatic action during the growing season. The contour furrows used with row crops are either large when the crop is planted and made smaller by cultivation or small after planting and made larger by cultivation, depending on the type of farming. Average conditions for the growing season are used in table 9.1. The relative effects of contouring for all croplands shown in the table are based on data from experimental watersheds having slopes from 3 to 8 percent. Stripcropping is a land use and treatment not specifically shown in table 9.1 because it is a composite of uses and treatments. It is evaluated by the method of example 10.5. The terraced entries in table 9.1 refer to systems containing open-end level or graded terraces, grassed-waterway outlets, and contour furrows between the terraces. The hydrologic effects are due to the replacement of a low-infiltration land use by grassed waterways and to the increased opportunity for infiltration in the furrows and terraces. Closed-end level terraces, not shown in table 9.1, are evaluated by the methods in chapter 12.

Grassland

Grassland in watersheds can be evaluated by means of the three hydrologic conditions of native pasture or range shown in table 8.1, which are based on cover effectiveness, not forage production. The percent of area covered (or density) and the intensity of grazing are visually estimated. In making the estimates keep in mind that grazing on any but dry soils will result in lowering of infiltration rates due to compaction of the soil by hooves, an effect that may carry over for a year or more even without further grazing.

An alternative system of evaluation is shown in table 8.2, in which density and air-dry weights of grasses and litter are used. The air-dry weights are determined by sampling. The field work can be kept to a minimum by sampling a small number of representative sites rather than a large number of random sites. In the table the classes with plus signs are midway between adjacent classes, so that the CN for these classes must be obtained by interpolation in table 9.1 or by the method shown in example 7.1.

Contour furrows on native pasture or range are longer lasting than those on cultivated land, their length of life being dependent on the soil, intensity of grazing, and on the density of cover. The dimensions and spacings of furrows vary with climate and topography. The CN in table 9.1 are based on data from contoured grassland watersheds in the central and southern Great Plains. Terraces are seldom used on grassland. When they are, the construction methods

Table 8.1.--Classification of native pasture or range

Vegetative condition	Hydrologic condition
Heavily grazed. Has no mulch or has plant cover on less than 1/2 of the area.	Poor
Not heavily grazed. Has plant cover on 1/2 to 3/4 of the area.	Fair
Lightly grazed. Has plant cover on more than 3/4 of the area.	Good

Table 8.2.--Air-dry weight classification of native pasture or range

Cover density (percent)	Plant and litter air-dry weight (tons per acre): Less than 0.5	0.5 to 1.5	More than 1.5
Less than 50	Poor	Poor +	Fair
50 to 75	Poor +	Fair	Fair +
More than 75	Fair	Fair +	Good

expose bare soils and for 2 or 3 years the terraced grassland is more like terraced cropland in its effect on surface runoff.

Meadow is a field on which grass is continuously grown, protected from grazing, and generally mowed for hay. Drained meadows (those having low water tables) have little or no surface runoff except during storms that have high rainfall intensities. Undrained meadows (those having high water tables) may be so wet as to be the equivalent of water surfaces in the runoff computations of chapter 10. If a wet meadow is drained, its soil-group classification as well as its land use and treatment class may change (see chapter 7 regarding the change in soil classification).

Woods and Forest

Woods are usually small isolated groves of trees being raised for farm

or ranch use. The woods can be evaluated as shown in table 8.3, which is based on cover effectiveness, not on timber production. The hydrologic condition is visually estimated.

In areas where National or commercial forest covers a large part of a watershed, the SCS hydrologist is guided by the memorandum of understanding between the Forest Service and the SCS. The Forest Service procedure for determining forest hydrologic conditions is given in chapter 4 of "Forest and Range Hydrology Handbook" U.S. Forest Service, Washington, D. C., April 1959. Excerpts from that handbook are given in chapter 9.

Determinations of Classes

The land use and treatment classes on a watershed can be determined at the same time the soils are classified (chap. 7). As with soils, the classes are determined for hydrologic units (chap. 6). Locations of the classes within the units are ignored. [1] A work sheet with classes shown in the order given in table 9.1 is convenient for tabulating percentages or acreages and is useful later in computing weighted CN as shown in chapter 10. It should take less than a day to classify the cover on a watershed of 400 square miles.

1/ For an analytical study of the effects of location of cover in a watershed on the shapes of outflow hydrographs, see the chapter by Merrill Bernard in "Headwaters Control and Use," U.S. Dept. of Agric., April 1937. Bernard's study shows that the percentage of area in high runoff producing crops has more influence on the hydrographs than does the location of these crops within the watershed. The effect of location is significant, however, when corn and grass are concentrated in equal-sized areas.

Table 8.3.--Classification of woods

Vegetative condition	Hydrologic condition
Heavily grazed or regularly burned. Litter, small trees, and brush are destroyed.	Poor
Grazed but not burned. There may be some litter but these woods are not protected.	Fair
Protected from grazing. Litter and shrubs cover the soil.	Good

* * * *

NATIONAL ENGINEERING HANDBOOK

SECTION 4

HYDROLOGY

CHAPTER 9. HYDROLOGIC SOIL-COVER COMPLEXES

by

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SCS NATIONAL ENGINEERING HANDBOOK

SECTION 4

HYDROLOGY

CHAPTER 9--HYDROLOGIC SOIL-COVER COMPLEXES

CONTENTS

CHAPTER 9. HYDROLOGIC SOIL-COVER COMPLEXES

A combination of a hydrologic soil group (soil) and a land use and treatment class (cover) is a hydrologic soil-cover complex. This chapter gives tables and graphs of runoff curve numbers (CN) assigned to such complexes. Its CN indicates the runoff potential of a complex during periods when the soil is not frozen, the higher a CN the higher a potential, and specifies which runoff curve of figure 10.1 is to be used in estimating runoff for the complex (chap. 10). Applications and further discussions of CN are given in chapters 10, 11, and 12.

Determinations of Complexes and CN

AGRICULTURAL LAND

Complexes and assigned CN for combinations of soil groups of chapter 7 and land use and treatment classes of chapter 8 are given in table 9.1. Also given are some complexes that make applications of the table more direct. Impervious and water surfaces, which are not listed, are always assigned a CN of 100.

ASSIGNMENT OF CN TO COMPLEXES. Table 9.1 was developed as follows. The data literature was searched for watersheds in single complexes (one soil group and one cover); watersheds were found for most of the listed complexes. An average CN for each watershed was obtained by the method of example 5.4, using rainfall-runoff data for storms producing the annual floods (chap. 18). The watersheds were generally less than 1 square mile in size, the number of watersheds for a complex varied, and the storms were of 1 day or less duration. The CN of watersheds in the same complex were averaged, all CN for a cover were plotted as shown in figure 7.2, a curve for each cover was drawn with greater weight given to CN based on data from more than one watershed, and each curve was extended as far as necessary to provide CN for ungaged complexes. All but the last three lines of

Table 9.1.--Runoff curve numbers for hydrologic soil-cover complexes

(Antecedent moisture condition II, and I_a = 0.2 S)

Land use	Cover: Treatment or practice	Cover: Hydrologic condition	Hydrologic soil group: A	B	C	D
Fallow	Straight row	----	77	86	91	94
Row crops	"	Poor	72	81	88	91
	"	Good	67	78	85	89
	Contoured	Poor	70	79	84	88
	"	Good	65	75	82	86
	"and terraced	Poor	66	74	80	82
	" " "	Good	62	71	78	81
Small grain	Straight row	Poor	65	76	84	88
		Good	63	75	83	87
	Contoured	Poor	63	74	82	85
		Good	61	73	81	84
	"and terraced	Poor	61	72	79	82
		Good	59	70	78	81
Close-seeded legumes 1/ or rotation meadow	Straight row	Poor	66	77	85	89
	" "	Good	58	72	81	85
	Contoured	Poor	64	75	83	85
	"	Good	55	69	78	83
	"and terraced	Poor	63	73	80	83
	"and terraced	Good	51	67	76	80
Pasture or range		Poor	68	79	86	89
		Fair	49	69	79	84
		Good	39	61	74	80
	Contoured	Poor	47	67	81	88
	"	Fair	25	59	75	83
	"	Good	6	35	70	79
Meadow		Good	30	58	71	78
Woods		Poor	45	66	77	83
		Fair	36	60	73	79
		Good	25	55	70	77
Farmsteads		----	59	74	82	86
Roads (dirt) 2/		----	72	82	87	89
(hard surface) 2/		---	74	84	90	92

1/ Close-drilled or broadcast.
2/ Including right-of-way.

Table 9.1A.--Runoff curve numbers for hydrologic soil-cover complexes for conservation tillage and residue management

(Antecedent moisture condition II, and $I_a = 0.2S$)

Land use	Cover: Treatment or practice	Cover: Hydrologic condition[1/]	Hydrologic soil group: A	B	C	D
Fallow	Conservation tillage	poor	76	85	90	93
	Conservation tillage	good	74	83	88	90
Row crops	Conservation tillage	poor	71	80	87	90
	Conservation tillage	good	64	75	82	85
	Contoured + conservation	poor	69	78	83	87
	tillage	good	64	74	81	85
	Contoured + terraces	poor	65	73	79	81
	+ conservation tillage	good	61	70	77	80
Small grain	Conservation tillage	poor	64	75	83	86
	Conservation tillage	good	60	72	80	84
	Contoured + conservation	poor	62	73	81	84
	tillage	good	60	72	80	83
	Contoured + terraces	poor	60	71	78	81
	+ conservation tillage	good	58	69	77	80

1/ For conservation tillage poor hydrologic condition, 5 to 20 percent of the surface is covered with residue (less than 750 #/acre row crops or 300 #/acre small grain).

For conservation tillage good hydrologic condition, more than 20 percent of the surface is covered with residue (greater than 750 #/acre row crops or 300 #/acre small grain).

NOTE: Percent cover should be estimated at the time of year storms occur.

CN entries in table 9.1 are taken from these curves. For the arbitrary complexes in the last three lines the proportions of different covers were estimated and CN computed from previously derived CN.

Table 9.1 has not been significantly changed since its construction in 1954 but supplementary tables for special regions have been developed. These tables are given later in this chapter.

USE OF TABLE 9.1. Chapters 7 and 8 describe how soils and cover of a watershed or other land area are classified in the field. After the classification is completed, CN are read from table 9.1 and applied as described in chapter 10. Because the principal use of CN is for estimating runoff from rainfall, the examples of applications are given in chapter 10.

NATIONAL AND COMMERCIAL FOREST: FOREST-RANGE

Chapter 4 of "Forest and Range Hydrology Handbook," U.S. Forest Service, Washington, D. C., 1959, describes how CN are determined for national and commercial forests in the eastern United States. Section 1 of "Handbook on Methods of Hydrologic Analysis," U.S. Forest Service, Washington, D. C., 1959, describes how CN are determined for forest-range regions in the western United States. Selections from these handbooks are given here to show the differences from SCS procedure; the handbooks should be consulted for details and examples.

Forest in Eastern United States

In the humid forest regions of the eastern United States, soil group, humus type, and humus depth are the principal factors used in the Forest Service method of determining CN. The undecomposed leaves or needles, twigs, bark, and other vegetative debris on the forest floor form the litter from which humus is derived. Litter protects humus from oxidation and therefore indirectly enters into the determination; if the depth of litter is less than 1/2 inch the humus is considered unprotected and the hydrologic condition class (fig. 9.1) is reduced by 0.5.

Humus is the organic layer immediately below the litter layer from which it is derived. It may consist of mull, which is an intimate mixture of organic matter and mineral soil, or of mor, which is practically pure organic matter unrecognizable as to origin from material lying on the forest floor. Humus depth increases with age

of forest stand until an equilibrium is reached between the processes that build up humus and those that break it down. As much as 12 inches of humus may be produced under favorable conditions, but a depth of 5 or 6 inches is considered the maximum attainable under average conditions. Under good management practices (proper use, protection, and improvement), humus is porous and has high infiltration and storage capacities. Under poor management practices (burning, overcutting, or overgrazing), humus is compact enough to impede the absorption of water.

Humus is evaluated by means of degrees of compaction, which are:

1. Compact. Mulls are firm; mors are felty.

2. Moderately compact. A transition stage.

3. Loose or friable. Mulls are not firm; mors are not felty.

Frost in compact humus is the concrete form, which inhibits infiltration, and in loose humus it is the granular or stalactite form, which does not. Because of the correlation between humus type and frost, a separate determination of the effects of frost is unnecessary.

The hydrologic condition of a forest area is the runoff-producing potential. The condition class is indicated by a number ranging from 1 to 6, the lower the number the higher the potential. The relation between classes and humus type and depth is shown in figure 9.1.

DETERMINATION OF CN FOR PRESENT HYDROLOGIC CONDITION. The CN for the present hydrologic condition of a forest area is determined as follows: sample plots are located in the area; soil group, litter depth, humus type, and humus depth are determined by means of shallow soil wells dug in the plots; the nomograph, figure 9.1, gives the hydrologic condition class of the plot; the network chart, figure 9.2, gives the CN. An average or weighted CN is obtained as described in chapter 10.

DETERMINATION OF CN FOR FUTURE HYDROLOGIC CONDITION. The CN for the future hydrologic condition of a forest area is determined from the improvement potential of the area, which is estimated by means of table 9.2. Definitions of terms used in the table are:

Improvement potential. The potential for improvement of the hydrologic condition of a site by proper use and treatment in the future. Physiography of the site enters into the determination of potential. The symbols for classes of potential are H = high, M = moderate, and L = low. A high potential means the most rapid rate of improvement, a low potential the slowest.

Table 9.2.--Physiographic factors and forest hydrologic-condition-improvement potential indexes

Aspect	Soil class	Soil depth	Slope position											
			Lower slope (streambank to one-fourth distance up slope)			One-fourth to one-half distance up slope			One-half to three-fourths distance up slope			Upper slope (three-fourths distance to top of slope)		
			Slope percent			Slope percent			Slope percent			Slope percent		
			0-20	21-40	41+	0-20	21-40	41+	0-20	21-40	41+	0-20	21-40	41+
		(inches)												
North to east	Clay	13-24	H	H	M	H	M	M	M	M	L	M	L	L
		25+	H	H	H	H	H	H	H	H	M	H	M	M
	Loam	13-24	H	H	H	H	H	M	H	M	M	M	M	L
		25+	H	H	H	H	H	H	H	H	H	H	H	M
	Sand	13+	H	M	M	M	M	L	M	L	L	L	L	L
South to west	Clay	13-24	M	M	L	M	L	L	L	L	L	L	L	L
		25+	H	M	M	M	M	L	M	L	L	L	L	L
	Loam	13-24	H	M	M	M	M	L	M	L	L	L	L	L
		25+	H	H	H	H	H	M	M	M	M	M	M	L
	Sand	13+	M	L	L	L	L	L	L	L	L	L	L	L
Northwest and southwest	Clay	13-24	H	M	L	M	M	L	M	L	L	L	L	L
		25+	H	H	H	H	M	M	H	M	L	M	M	L
	Loam	13-24	H	H	M	H	M	M	M	M	L	M	L	L
		25+	H	H	H	H	H	H	H	H	M	H	M	M
	Sand	13+	M	L	L	M	L	L	L	L	L	L	L	L

This is table 4.1 in U.S. Forest Service "Forest and Range Hydrology Handbook."

<u>Aspect</u>. A compass reading to the nearest octant, taken from the center of the sample plot and looking downslope on a line at right angles to the contours.

<u>Soil class</u>. Texture of the mineral soil immediately below the humus layer if any. Note that these classes differ from the soil groups of chapter 7 because the classes are concerned with forest growth, the groups with runoff.

<u>Soil depth</u>. A determination made in the sample plot. Rock outcrops or soils less than 13 inches deep are put in the 13- to 24-inch class.

<u>Slope</u>. A percentage reading of land slope, taken at the center of the plot.

<u>Slope position</u>. A forest growth class based on the vertical position of the plot relative to a stream (fig. 9.3).

Once the improvement potential is known, the time period for achieving the potential is estimated on the basis of use and treatment to be given the area; consideration is given to measures for protection from fire, overgrazing, overcutting, damaging logging, and epidemics of insects or diseases, to tree planting in open fields or woods openings, and to stand improvement. The CN for the area is estimated using figure 9.4, as illustrated in the following example.

> Example 9.1.--A forest area has a present hydrologic condition class of 1.3 and soils in the A group. The improvement potential is high and it is estimated that a 50-year period is necessary to bring the area to this level. Determine the future CN for the area.
>
> 1. Determine the present CN. Enter figure 9.2 with the hydrologic condition class of 1.3 and at the line for soil group A read a CN of 54.
>
> 2. Determine the future hydrologic condition class. Enter figure 9.4 with the present class of 1.3, go across to the curve for high potential, and read 6 years on the time scale. To this value add one-half the improvement period: 6 + (50/2) = 31 years, follow the "high" curve to its intersection with 31 years on the time scale, and read a future class of 3.4. This estimate is based on 100 percent accomplishment of recommended use and treatment; if less accomplishment is expected, the condition class is proportionately reduced.
>
> 3. Determine the future CN. Enter figure 9.2 with the future class of 3.4 and at the line for soil group A read a CN of 37.

Forest-Range in Western United States

In the forest-range regions of the western United States, soil group, cover type, and cover density are the principal factors used in estimating CN. Figures 9.5 and 9.6 show the relationships between these factors and CN for soil-cover complexes used to date. The figures are based on information in table 2.1, part 2, of the Forest Service "Handbook on Methods of Hydrologic Analysis." The covers are defined as follows:

Herbaceous.--Grass-weed-brush mixtures with brush the minor element.

Oak-Aspen.-- Mountain brush mixtures of oak, aspen, mountain mahogany, bitter brush, maple, and other brush.

Juniper-Grass.--Juniper or piñon with an understory of grass.

Sage-Grass.--Sage with an understory of grass.

The amount of litter is taken into account when estimating the density of cover.

Present hydrologic conditions are determined from existing surveys or by reconnaissance, and future conditions from the estimate of cover and density changes due to proper use and treatment.

SUPPLEMENTARY TABLES OF CN

Tables 9.3, 9.4, and 9.5 are supplements to table 9.1 and are used in the same way.

Table 9.3 gives CN for selected covers in Puerto Rico. The CN were obtained using a relation between storm and annual data and the annual rainfall-runoff data for experimental plots at Mayaguez.

Table 9.4 gives CN for complexes in a typical watershed in Contra Costa County, California. The CN were obtained by the Contra Costa County Flood Control District and SCS, using streamflow data from the watershed and a trial-and-error process. The range in CN for a particular cover and soil group indicates the variation for soil subgroups.

Table 9.5 gives CN for sugarcane complexes in Hawaii. The CN are tentative estimates now undergoing study. Degrees of cover in the table are defined as follows:

Table 9.3.--Runoff curve numbers for hydrologic soil-cover complexes in Puerto Rico (antecedent moisture condition II, and I_a = 0.2 S).

Cover and condition	Hydrologic soil group			
	A	B	C	D
Fallow	77	86	91	93
Grass (bunch grass, or poor stand of sod)	51	70	80	84
Coffee (no ground cover, no terraces)	48	68	79	83
Coffee (with ground cover and terraces)	22	52	68	75
Minor crops (garden or truck crops)	45	66	77	83
Tropical kudzu	19	50	67	74
Sugarcane (trash burned; straight-row)	43	65	77	82
Sugarcane (trash mulch; straight row)	45	66	77	83
Sugarcane (in holes; on contour)	24	53	69	76
Sugarcane (in furrows; on contour)	32	58	72	79

Table 9.4.--Runoff curve numbers for hydrologic soil-cover complexes of a typical watershed in Contra Costa County, California (antecedent moisture condition II, and I_a = 0.2 S).

Cover	Condition	Hydrologic soil group			
		A	B	C	D
Scrub (native brush)	----	25-30	41-46	57-63	66
Grass-oak (native oaks with understory of forbs and annual grasses)	Good	29-33	43-48	59-65	67
Irrigated pasture	Good	32-37	46-51	62-68	70
Orchard (winter period with understory of cover crop)	Good	37-41	50-55	64-69	71
Range (annual grass)	Fair	46-49	57-60	68-72	74
Small grain (contoured)	Good	61-64	69-71	76-80	81
Truck crops (straight-row)	Good	67-69	74-76	80-83	84
Urban areas:					
Low density (15 to 18 percent impervious surfaces)		69-71	75-78	82-84	86
Medium density (21 to 27 percent impervious surfaces)		71-73	77-80	84-86	88
High density (50 to 75 percent impervious surfaces)		73-75	79-82	86-88	90

Table 9.5.--Runoff curve numbers; tentative estimates for sugarcane hydrologic soil-cover complexes in Hawaii (antecedent moisture condition II, and $I_a = 0.2\ S$).

Cover and treatment	Hydrologic soil group			
	A	B	C	D
Sugarcane:				
Limited cover, straight row	67	78	85	89
Partial cover, straight row	49	69	79	84
Complete cover, straight row	39	61	74	80
Limited cover, contoured	65	75	82	86
Partial cover, contoured	25	59	75	83
Complete cover, contoured	6	35	70	79

Limited cover.--Cane newly planted, or ratooned cane with a limited root system; canopy over less than 1/2 the field area.

Partial cover.--Cane in the transition period between limited and complete cover; canopy over 1/2 to nearly the entire field area.

Complete cover.--Cane from the stage of growth when full canopy is provided to the stage at harvest.

Straight-row planting is up and down hill or cross-slope on slopes greater than 2 percent. Contoured planting is the usual contouring or cross-slope planting on slopes less than 2 percent.

* * * *

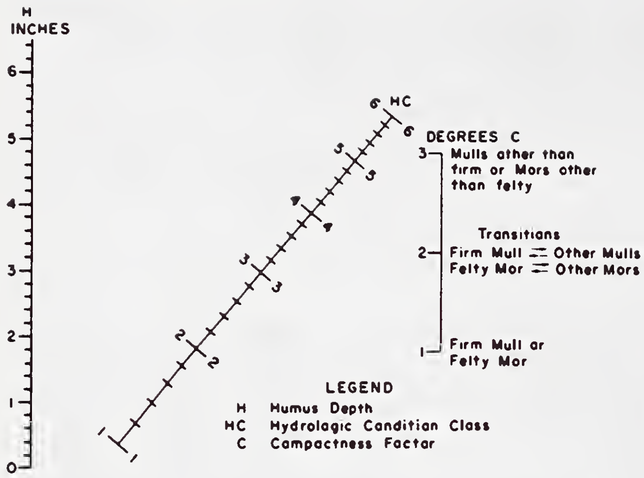

FIGURE 9.1 PRESENT HYDROLOGIC CONDITION OF FOREST AND WOODLAND

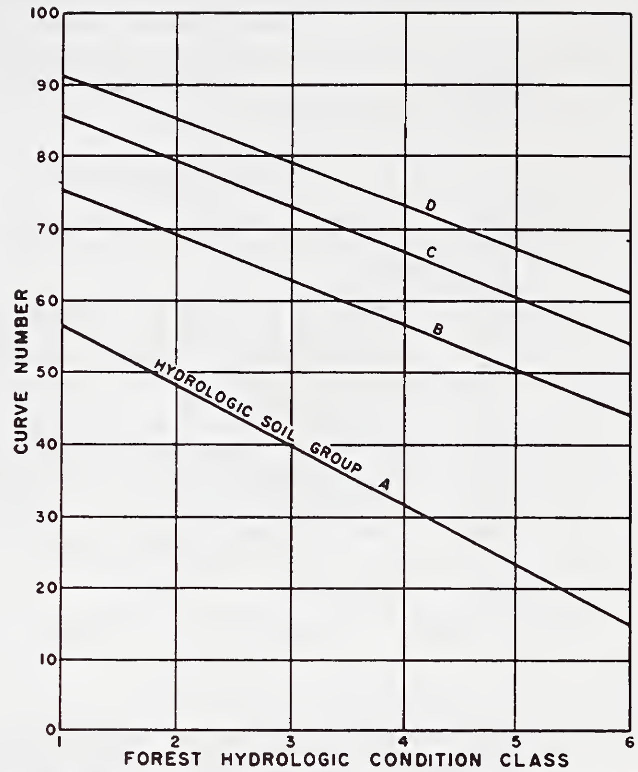

FIGURE 9.2 CURVE NUMBERS BY HYDROLOGIC SOIL GROUP AND FOREST HYDROLOGIC CONDITION CLASSES

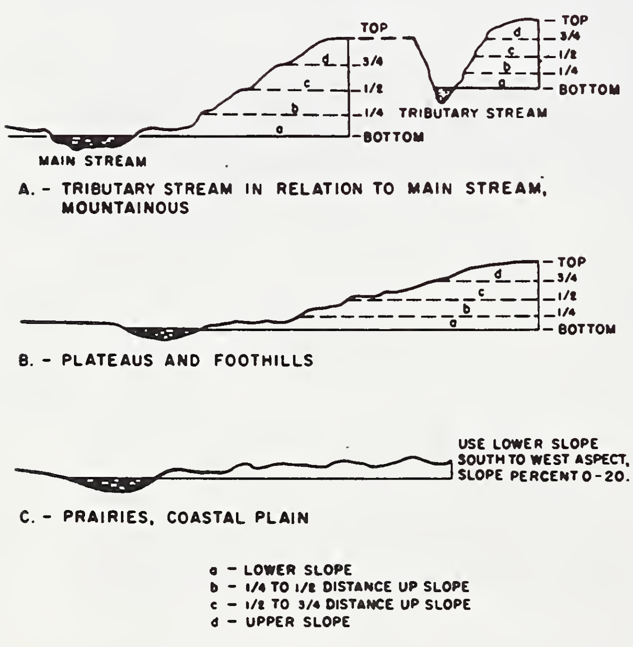

FIGURE 9.3 - EXAMPLES OF SLOPE POSITION

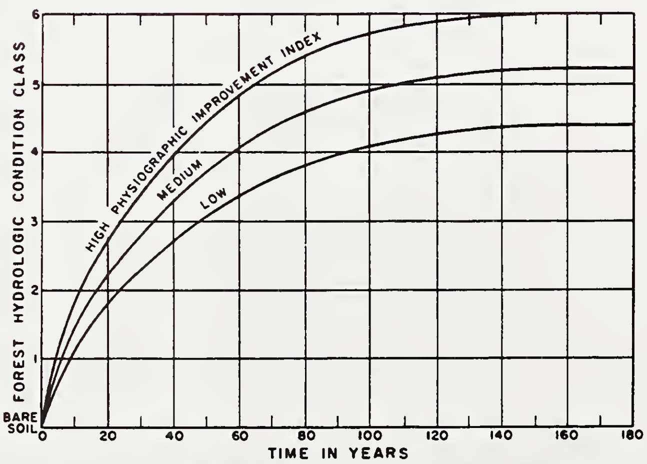

FIGURE 9.4 RATE OF IMPROVEMENT OF FOREST HYDROLOGIC CONDITION UNDER MANAGEMENT. STARTING CONDITION - BARE SOIL

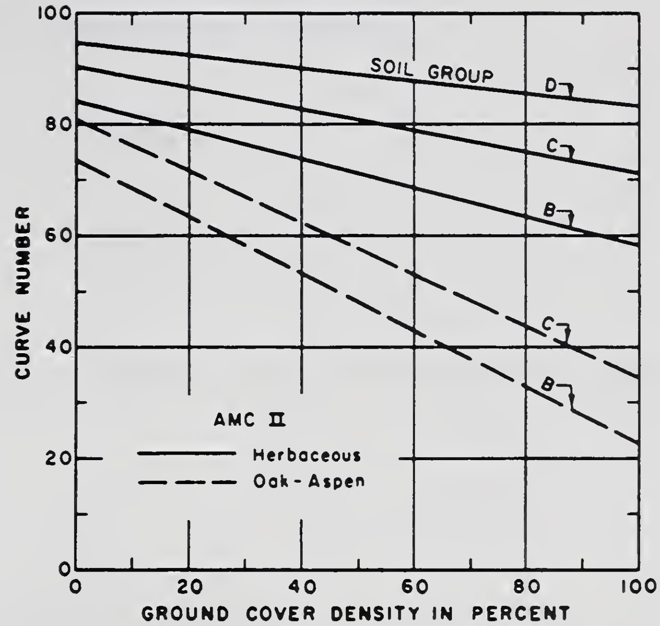

Figure 9.5.--Graph for estimating runoff curve numbers of forest-range complexes in western United States: herbaceous and oak-aspen complexes.

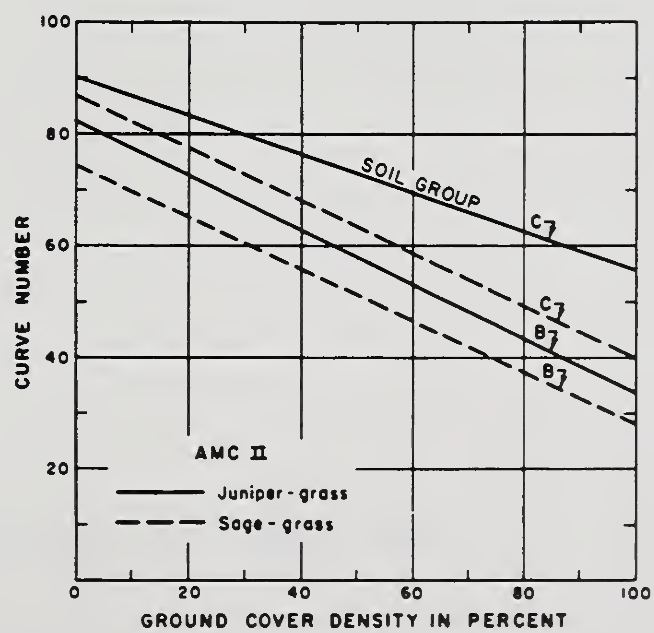

Figure 9.6.--Graph for estimating runoff curve numbers of forest-range complexes in western United States: juniper-grass and sage-grass complexes.

NATIONAL ENGINEERING HANDBOOK

SECTION 4

HYDROLOGY

CHAPTER 10. ESTIMATION OF DIRECT RUNOFF FROM STORM RAINFALL

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SCS NATIONAL ENGINEERING HANDBOOK

SECTION 4

HYDROLOGY

CHAPTER 10--ESTIMATION OF DIRECT RUNOFF FROM STORM RAINFALL

CONTENTS

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CHAPTER 10. ESTIMATION OF DIRECT RUNOFF FROM STORM RAINFALL

The SCS method of estimating direct runoff from storm rainfall is described in this chapter. The rainfall-runoff relation of the method is developed, parameters in the relation are discussed, and applications of the method are illustrated by examples.

Introduction

The SCS method of estimating direct runoff from storm rainfall is based on methods developed by SCS hydrologists in the last three decades, and it is in effect a consolidation of these earlier methods. The hydrologic principles of the method are not new, but they are put to new uses. Because most SCS work is with ungaged watersheds (not gaged for runoff) the method was made to be usable with rainfall and watershed data that are ordinarily available or easily obtainable for such watersheds. If runoff data are also available the method is adaptable to their use as illustrated in chapter 5.

The principal application of the method is in estimating quantities of runoff in flood hydrographs or in relation to flood peak rates (chap. 16). These quantities consist of one or more types of runoff. An understanding of the types is necessary to apply the method properly in different climatic regions. The classification of types used in this handbook is based on the time from the beginning of a storm to the time of the appearance of a type in the hydrograph. Four types are distinguished:

Channel runoff occurs when rain falls on a flowing stream or on the impervious surfaces of a streamflow-measuring installation. It appears in the hydrograph at the start of the storm and continues throughout it, varying with the rainfall intensity. It is generally a negligible quantity in flood hydrographs, and no attention is given to it except in special studies (see the discussion concerning the relationship of I_a to S in figure 10.2).

Surface runoff occurs only when the rainfall rate is greater than the infiltration rate. The runoff flows on the watershed surface to the point of reference. This type appears in the hydrograph after the initial demands of interception, infiltration, and surface storage have been satisfied. It varies during the storm and ends during or soon after it. Surface runoff flowing down dry channels of watersheds in arid, semiarid, or subhumid climates is reduced by transmission losses (chap. 19), which may be large enough to eliminate the runoff entirely.

Subsurface flow occurs when infiltrated rainfall meets an underground zone of low transmission, travels above the zone to the soil surface downhill, and appears as a seep or spring. This type is often called "quick return flow" because it appears in the hydrograph during or soon after the storm.

Base flow occurs when there is a fairly steady flow from natural storage. The flow comes from lakes or swamps, or from an aquifer replenished by infiltrated rainfall or surface runoff, or from "bank storage", which is supplied by infiltration into channel banks as the stream water level rises and which drains back into the stream as the water level falls. This type seldom appears soon enough after a storm to have any influence on the rates of the hydrograph for that storm, but base flow from a previous storm will increase the rates. Base flow must be taken into account in the design of the principal spillway of a floodwater-retarding structure (chap. 21).

All types do not regularly appear on all watersheds. Climate is one indicator of the probability of the types. In arid regions the flow on smaller watersheds is nearly always surface runoff, but in humid regions it is generally more of the subsurface type. But a long succession of storms produces subsurface or base flow even in dry climates although the probability of this occurring is less in dry climates than in wet climates.

In flood hydrology it is customary to deal separately with base flow and to combine all other types into direct runoff, which consists of channel runoff, surface runoff, and subsurface flow in unknown proportions. The SCS method estimates direct runoff, but the proportions of surface runoff and subsurface flow (channel runoff is ignored) can be appraised by means of the runoff curve number (CN), which is another indicator of the probability of flow types: the larger the CN the more likely that the estimate is of surface runoff. This principle is also employed for estimating watershed lag as shown in figure 15.3. The rainfall-runoff relation of the SCS method can be made to operate with a particular type of flow; it was linked with direct runoff, as described in chapter 9, for the convenience of applications.

The Rainfall-Runoff Relation

The most generally available rainfall data in the United States are the amounts measured at nonrecording rain gages, and it was for the use of such data or their equivalent that the rainfall-runoff relation was developed. The data are totals for one or more storms occurring in a calendar day, and nothing is known about the time distributions. The relation therefore excludes time as a variable; this means that rainfall intensity is ignored. If everything but storm duration or intensity is the same for two storms, the estimate of runoff is the same for both storms. Runoff amounts for specified time increments of a storm can be estimated as shown in example 10.6, but even in this process the effect of rainfall intensity is ignored.

DEVELOPMENT

If records of natural rainfall and runoff for a large storm over a small area are used, plotting of accumulated runoff versus accumulated rainfall will show that runoff starts after some rain accumulates (there is an "initial abstraction" of rainfall) and that the double-mass line curves, becoming asymptotic to a straight line. On arithmetic graph paper and with equal scales, the straight line has a 45-degree slope. The relation between rainfall and runoff can be developed from this plotting, but a better explanation of the relation is given by first studying a storm in which rainfall and runoff begin simultaneously (initial abstraction is zero). For the simpler storm the relation between rainfall, runoff, and retention (the rain not converted to runoff) at any point on the mass curve can be expressed as:

$$\frac{F}{S} = \frac{Q}{P} \qquad (10.1)$$

where:

- F = actual retention after runoff begins
- S = potential maximum retention after runoff begins ($S \geq F$)
- Q = actual runoff
- P = rainfall ($P \geq Q$)

Equation 10.1 applies to on-site runoff; for large watersheds there is a lag in the appearance of the runoff at the stream gage, and the double-mass curve produces a different relation. But if storm totals for P and Q are used equation 10.1 does apply even for large watersheds because the effects of the lag are removed.

The retention, S, is a constant for a particular storm because it is the maximum that can occur under the existing conditions if the storm continues without limit. The retention F varies because it is the difference between P and Q at any point on the mass curve, or:

$$F = P - Q \tag{10.2}$$

Equation 10.1 can therefore be rewritten:

$$\frac{P - Q}{S} = \frac{Q}{P} \tag{10.3}$$

Solving for Q produces the equation:

$$Q = \frac{P^2}{P + S} \tag{10.4}$$

which is a rainfall-runoff relation in which the initial abstraction is zero.

If an initial abstraction (I_a) greater than zero is considered, the amount of rainfall available for runoff is $P - I_a$ instead of P. By substituting $P - I_a$ for P in equations 10.1 through 10.4 the following equations result. The equivalent of equation 10.1 becomes:

$$\frac{F}{S} = \frac{Q}{P - I_a} \tag{10.5}$$

where $F \leq S$, and $Q \leq (P - I_a)$. The total retention for a storm consists of I_a and F. The total potential maximum retention (as P gets very large) consists of I_a and S.

Equation 10.2 becomes:

$$F = (P - I_a) - Q \tag{10.6}$$

equation 10.3 becomes:

$$\frac{(P - I_a) - Q}{S} = \frac{Q}{(P - I_a)} \tag{10.7}$$

and equation 10.4 becomes:

$$Q = \frac{(P - I_a)^2}{(P - I_a) + S} \tag{10.8}$$

which is the rainfall-runoff relation with the initial abstraction taken into account.

The initial abstraction consists mainly of interception, infiltration, and surface storage, all of which occur before runoff begins. The insert on figure 10.1 shows the position of I_a in a typical storm. To remove the necessity for estimating these variables in equation 10.8, the relation between I_a and S (which includes I_a) was developed by means of rainfall and runoff data from experimental small watersheds. The relation is discussed later in connection with figure 10.2. The empirical relationship is:

$$I_a = 0.2\ S \qquad (10.9)$$

Substituting 10.9 in 10.8 gives:

$$Q = \frac{(P - 0.2\ S)^2}{P + 0.8\ S} \qquad (10.10)$$

which is the rainfall-runoff relation used in the SCS method of estimating direct runoff from storm rainfall.

Retention Parameters

Using the equation 10.9 relationship, the total maximum retention can be expressed as 1.2 S. I_a, as previously stated, consists mainly of interception, infiltration, and surface storage occurring before runoff begins. S is mainly the infiltration occurring after runoff begins. This later infiltration is controlled by the rate of infiltration at the soil surface or by the rate of transmission in the soil profile or by the water-storage capacity of the profile, whichever is the limiting factor. A succession of storms, such as one a day for a week, reduces the magnitude of S each day because the limiting factor does not have the opportunity to completely recover its rate or capacity through weathering, evapotranspiration, or drainage. But there is enough recovery, depending on the soil-cover complex, to limit the reduction. During such a storm period the magnitude of S remains virtually the same after the second or third day even if the rains are large so that there is, from a practical viewpoint, a lower limit to S for a given soil-cover complex. Similarly, there is a practical upper limit to S, again depending on the soil-cover complex, beyond which the recovery cannot take S unless the complex is altered.

In the SCS method, the change in S (actually in CN) is based on an antecedent moisture condition (AMC) determined by the total rainfall in the 5-day period preceding a storm. Three levels of AMC are used: AMC-I is the lower limit of moisture or the upper limit of S, AMC-II is the average for which the CN of table 9.1 apply, and AMC-III is the upper limit of moisture or the lower limit of S. The CN in table 9.1 were determined by means of rainfall-runoff plottings as described in chapter 9. The same plottings served for getting CN for AMC-I and AMC-III. That is, the curves of figure 10.1, when superimposed on a plotting, also showed which curves best fit the highest (AMC-III) and lowest (AMC-I) thirds of the plotting. The CN for high and low moisture levels were empirically related to the CN of table 9.1; the results are shown in columns 1, 2, and 3 of table 10.1, which also gives values of S and I_a for the CN in column 1. The rainfall amounts on which the selection of AMC is based are given in table 4.2; the discussion in chapter 2 concerns the value of rainfall alone as a criterion for AMC. Use of tables 4.2 and 10.1 is demonstrated later in this chapter. In the section on comparisons of computed and actual runoffs, an example shows that for certain problems the extreme AMC can be ignored and the average CN of table 9.1 alone applied.

<u>RELATION OF I_a TO S</u>. Equation 10.9 is based on the results shown in figure 10.2 which is a plotting of I_a versus S for individual storms. The data were derived from records of natural rainfall and runoff from watersheds less than 10 acres in size. The large amount of scatter in the plotting is due mainly to errors in the estimates of I_a. The magnitudes of S were estimated by plotting total storm rainfall and runoff on figure 10.1, determining the CN, and determining the S from table 10.1. The magnitudes of I_a were estimated by taking the accumulated rainfall from the beginning of a storm to the time when runoff started. Errors in S were due to determinations of average watershed rainfall totals; these errors were very small. Errors in I_a were due to one or more of the following: (i) difficulty of determining the time when rainfall began, because of storm travel and lack of instrumentation, (ii) difficulty of determining the time when runoff began, owing to the effects of rain on the measuring installations (channel runoff) and to the lag of runoff from the watersheds, and (iii) impossibility of determining how much interception prior to runoff later made its way to the soil surface and contributed to runoff; the signs and magnitudes of these errors are not known. Only enough points are plotted in figure 10.2 to show the variability of the data. The line of relationship cuts the plotting into two equal numbers of points, and the slope of the line is 1:1 because the data do not indicate otherwise. A significant statistical correlation (chap. 18) between I_a and S can be made by adding more points and increasing the "degrees of freedom," but the standard error of estimate will remain large owing to the deficiencies in the data.

Graphs and Tables for the Solution of Equation 10.10

Sheets 1 and 2 of figure 10.1 contain graphs for the rapid solution of equation 10.10. The parameter CN (runoff curve number or hydrologic soil-cover complex number) is a transformation of S, and it is used to make interpolating, averaging, and weighting operations more nearly linear. The transformation is:

$$CN = \frac{1000}{S + 10} \qquad (10.11)$$

or

$$S = \frac{1000}{CN} - 10 \qquad (10.12)$$

Tables for the solution of equation 10.10 are given in SCS Technical Release 16 for P from zero to 40.9 inches by steps of 0.1-inch and for all whole-numbered CN in the range from 55 through 98.

USE OF S AND CN. It is more convenient to use CN on figure 10.1, but it will generally be necessary to use S for other applications such as the analysis of runoff data or the development of supplementary runoff relationships. Example 5.5 and figure 5.6(b) illustrate a typical use of S. The relationship is developed using S, but a scale for CN is added later to the graph for ease of application.

Table 10.1. Curve numbers (CN) and constants for the case $I_a = 0.2$ S

1	2	3	4	5	1	2	3	4	5
CN for condition II	CN for conditions I	III	S values*	Curve* starts where P =	CN for condition II	CN for conditions I	III	S values*	Curve* starts where P =
			(inches)	(inches)				(inches)	(inches)
100	100	100	0	0	60	40	78	6.67	1.33
99	97	100	.101	.02	59	39	77	6.95	1.39
98	94	99	.204	.04	58	38	76	7.24	1.45
97	91	99	.309	.06	57	37	75	7.54	1.51
96	89	99	.417	.08	56	36	75	7.86	1.57
95	87	98	.526	.11	55	35	74	8.18	1.64
94	85	98	.638	.13	54	34	73	8.52	1.70
93	83	98	.753	.15	53	33	72	8.87	1.77
92	81	97	.870	.17	52	32	71	9.23	1.85
91	80	97	.989	.20	51	31	70	9.61	1.92
90	78	96	1.11	.22	50	31	70	10.0	2.00
89	76	96	1.24	.25	49	30	69	10.4	2.08
88	75	95	1.36	.27	48	29	68	10.8	2.16
87	73	95	1.49	.30	47	28	67	11.3	2.26
86	72	94	1.63	.33	46	27	66	11.7	2.34
85	70	94	1.76	.35	45	26	65	12.2	2.44
84	68	93	1.90	.38	44	25	64	12.7	2.54
83	67	93	2.05	.41	43	25	63	13.2	2.64
82	66	92	2.20	.44	42	24	62	13.8	2.76
81	64	92	2.34	.47	41	23	61	14.4	2.88
80	63	91	2.50	.50	40	22	60	15.0	3.00
79	62	91	2.66	.53	39	21	59	15.6	3.12
78	60	90	2.82	.56	38	21	58	16.3	3.26
77	59	89	2.99	.60	37	20	57	17.0	3.40
76	58	89	3.16	.63	36	19	56	17.8	3.56
75	57	88	3.33	.67	35	18	55	18.6	3.72
74	55	88	3.51	.70	34	18	54	19.4	3.88
73	54	87	3.70	.74	33	17	53	20.3	4.06
72	53	86	3.89	.78	32	16	52	21.2	4.24
71	52	86	4.08	.82	31	16	51	22.2	4.44
70	51	85	4.28	.86	30	15	50	23.3	4.66
69	50	84	4.49	.90					
68	48	84	4.70	.94	25	12	43	30.0	6.00
67	47	83	4.92	.98	20	9	37	40.0	8.00
66	46	82	5.15	1.03	15	6	30	56.7	11.34
65	45	82	5.38	1.08	10	4	22	90.0	18.00
64	44	81	5.62	1.12	5	2	13	190.0	38.00
63	43	80	5.87	1.17	0	0	0	infinity	infinity
62	42	79	6.13	1.23					
61	41	78	6.39	1.28					

*For CN in column 1.

Applications

The examples in this part mainly illustrate the use of tables 4.2, 9.1, and 10.1 and figure 10.1. Records from gaged watersheds are used in some examples to compare computed with actual runoffs. The errors in a runoff estimate are due to one or more of the following: empiricisms of table 4.2 or figure 4.9, or table 9.1 and similar tables in chapter 9, of the relation between AMC (columns 1, 2, and 3 of table 10.1), and of equation 10.9; and errors in determinations of average watershed rainfall (chap. 4), soil groups, (chap. 7), land use and treatment (chap. 8), and related computations. Consequently it is impossible to state a standard error of estimate for equation 10.10; comparisons of computed and actual runoffs indicate only the algebraic sums of errors from various sources.

<u>SINGLE STORMS</u>. The first example is a typical routine application of the estimation method when there is no question regarding the accuracy of rainfall, land use and treatment, and soil group determinations.

> Example 10.1.- During a storm an average depth of 4.3 inches of rain fell over a watershed with a cover of good pasture, soils in the C group, and an AMC-II. Estimate the direct runoff.
>
> 1. Determine the CN. In table 9.1 at "Pasture, good" and under soil group C read a CN of 74, which is for AMC-II.
>
> 2. Estimate the runoff. Enter figure 10.1 with the rainfall of 4.3 inches and at CN = 74 (by interpolation) find Q = 1.83 inches.

In practice the estimate of Q is carried to two decimal places to avoid confusing different estimates. Except for such needs the estimate should generally be rounded to one decimal place; in example 10.1 the rounded estimate is 1.8 inches. If the storm rainfall amount is not accurately known the estimate is rounded even further or the range of the estimate is given as in the following example.

> Example 10.2.--During a thunderstorm a rain of 6.0 inches was measured at a rain gage 5.0 miles from the center of a watershed that had a flood from this storm. The drainage area of the watershed is 840 acres, cover is fair pasture, soils are in the D group, and AMC-II applies. Estimate the direct runoff.

1. Determine the average watershed rainfall. Enter figure 4.4 with the distance of 5.0 miles and at line for a rain of 6.0 inches read a plus-error of 2.8 inches. The minus-error is half this, or 1.4 inches. The watershed is small enough that no "areal correction" of rainfall is necessary (see figure 21.-- and related discussion in chapter 21), therefore the average watershed rainfall ranges from 8.8 to 4.6 inches.

2. Determine the CN. In table 9.1 the CN is 84 for fair pasture in the D soil group.

3. Estimate the direct runoff. Enter figure 10.1 with the rainfall of 8.8 inches and at CN = 84 (by interpolation) read an estimated runoff of 6.87 inches; also enter with the rainfall of 4.6 inches and read a runoff of 2.91 inches. After rounding, the estimate of direct runoff is given as being between 2.9 and 6.9 inches or, better yet, between 3 and 7 inches. The probability level of figure 4.4 can also be used with the runoff estimate.

Table 10.1 is used when it is necessary to estimate runoff for a watershed in a dry or wet condition before a storm:

Example 10.3.--For the watershed of example 10.1, estimate the direct runoff for AMC-I and AMC-III and compare with the estimate for AMC-II.

1. Determine the CN for AMC-II. This is done in step 1 of example 10.1; the CN is 74.

2. Determine CN for other AMC. Enter table 10.1 at CN = 74 in column 1 and in columns 2 and 3 read CN = 55 for AMC-I and CN = 88 for AMC-III.

3. Estimate the runoffs. Enter figure 10.1 with the rainfall of 4.3 inches (from ex. 10.1) and at CN = 55, 74, and 88 read (by interpolation as necessary) that Q = 0.65, 1.83, and 3.00 inches, respectively. The comparison in terms of AMC-II runoff is as follows:

AMC	CN	Direct runoff, Q		
		Inches	As percent of rainfall	As percent of Q for AMC-II
I	55	0.65	15.1	35.6
II	74	1.83	42.5	100
III	88	3.00	69.8	164

Note that the runoff in inches or percents is not simply proportional to the CN so that the procedure does not allow for a short cut.

ALTERNATE METHODS OF ESTIMATION FOR MULTIPLE COMPLEXES. The direct runoff for watersheds having more than one hydrologic soil-cover complex can be estimated in either of two ways: in example 10.4 the runoff is estimated for each complex and weighted to get the watershed estimate; in example 10.5 the CN are weighted to get a watershed CN and the runoff is estimated using it.

Example 10.4.--A watershed of 630 acres has 400 acres in "Row crop, contoured, good rotation" and 230 acres in "Rotation meadow, contoured, good rotation." All soils are in the B group. Find the direct runoff for a rain of 5.1 inches when the watershed is in AMC-II.

1. Determine the CN. Table 9.1 shows that the CN are 75 for the row crop and 69 for the meadow.

2. Estimate runoff for each complex. Enter figure 10.1 with the rain of 5.1 inches and at CN of 75 and 69 read Q's of 2.52 and 2.03 inches respectively.

3. Compute the weighted runoff. The following table shows the work.

Hydrologic soil-cover complex	Acres	Q(inches)	Acres X Q
Row crop etc.	400	2.52	1,008
Meadow etc.	230	2.03	467
Totals:	630		1,475

The weighted Q is 1475/630 = 2.34 inches.

Example 10.5.--Use the watershed and rain data of example 10.4 and make the runoff estimate using a weighted CN.

1. Determine the CN. Table 9.1 shows that the CN are 75 for the row crop and 69 for the meadow.

2. Compute the weighted CN. The following table shows the work.

Hydrologic soil-cover complex	Acres	CN	Acres X CN
Row crop etc.	400	75	30,000
Meadow etc.	230	69	15,870
Totals:	630		45,870

The weighted CN is 45,870/630 = 72.8. Use 73.

3. Estimate the runoff. Enter figure 10.1 with the rain of 5.1 inches and at CN = 73 (by interpolation) read Q = 2.36 inches. (Note: Q is 2.34 inches just as in example 10.4 if the unrounded CN is used.)

Without the rounding in step 2 of example 10.5, both methods of weighting give the same Q to three significant figures, and there appears to be no reason for choosing one method over the other. But each method has its advantages and disadvantages. The method of weighted-Q always gives the correct result (in terms of the given data) but it required more work than the weighted-CN method especially when a watershed has many complexes. The method of weighted-CN is easier to use with many complexes or with a series of storms, but when there are large differences in CN for a watershed this method will under- or over-estimate Q, depending on the size of the storm rainfall. For example an urban watershed with 20 acres of impervious area (CN = 100) and 175 acres of lawn classed as good pasture on a B soil (CN = 61) will have the following Q's by the two methods (all entries in inches):

Storm rainfall:	1	2	4	8	16	32
Q (weighted-Q method):	0.10	0.27	1.14	3.91	10.85	26.10
Q (weighted-CN method):	0	.13	1.03	3.89	10.97	26.34

This comparison shows that the method of weighted-Q is preferable when small rainfalls are used and there are two or more widely differing CN on a watershed. For conditions other than these the method of weighted-CN is less time-consuming and almost as accurate.

MULTIPLE-DAY STORMS AND STORM SERIES. Data from a gaged small watershed will be used in the following example to illustrate (i) an application of the method of estimation to a storm series such as used in evaluation of a floodwater-retarding project, (ii) treatment of multiple-day storms, which differs from that of design storms in chapter 21, and (iii) the amount of error generally to be expected from use of the method. The data to be used are taken from:

Reference 1. "The Agriculture, Soils, Geology, and Topography of the Blacklands Experimental Watershed, Waco, Texas," Hydrologic Bulletin 5, U.S. Soil Conservation Service, 1942.

Reference 2. "Summary of Rainfall and Runoff, 1940-1951, at Blacklands Experimental Watershed, Waco, Texas," U.S. Soil Conservation Service, 1952.

The watershed is W-1 with an area of 176 acres, average annual rainfall of 34.95 inches for the period 1940-1952 inclusive, and average

storm rainfall depths determined from amounts at four gages on or very near the watershed. According to figure 4.6 (its scales must be extended for so small a watershed) the storm rainfall amounts will have a negligible error. With this exception the data to be used are equivalent to those ordinarily obtained for ungaged watersheds.

Example 10.6.--Estimate the runoff amounts from storms that produced the maximum annual peak rates of flow at watershed W-1, Waco, Texas, for the period 1940-1952 inclusive.

1. Determine the soil groups. Reference 1 shows that the soils are Houston Black Clay or equivalents. Table 7.1 in chapter 7 shows these soils are in the D group.

2. Determine the average land use and treatment for the period 1940-1952. Reference 2 gives information from which the average land use and treatment is determined to be:

Land use and treatment	Percent of area
Row crop, straight row, poor rotation	58
Small grain, straight row, poor rotation	25
Pasture (including hay), fair condition	15
Farmsteads and roads	2

3. Tabulate the storm dates, total rainfall for each date, and the 5-day antecedent rainfall. Reference 2 gives the information shown in columns 1 through 5 of table 10.2.

4. Determine the CN for AMC-I, -II, and -III. Table 9.1 gives the CN for each complex; the computation of the weighted CN for AMC-II is:

Hydrologic soil-cover complex	Percent/100	CN	Product
Row crop etc.	0.58	91	52.7
Small grain etc.	.25	88	22.0
Pasture etc.	.15	84	12.6
Farmsteads etc.	.02	94	1.9
Totals	1.00		89.2

No division of the product is necessary because "percent/100" is used. The CN is rounded to 89. CN for the other two AMC are obtained from table 10.1 and are:

AMC:	I	II	III
CN:	76	89	96

Table 10.2.--Working table for a storm series.

Year	Month	Day	Storm rainfall	Antecedent rainfall	AMC	CN	Estimated runoff By days	Estimated runoff Storm totals	Actual runoff By days	Actual runoff Storm totals	Differences By days	Differences Storm totals
			(in.)	(in.)			(in.)	(in.)	(in.)	(in.)	(in.)	(in.)
1940	Nov.	22	4.74	0.18	I	76	2.32		2.32		0	
		23	2.20		III	96	1.77		2.02		- .25	
		24	2.03		III	96	1.61		1.39		.22	
		25	.38		III	96	.13	5.83	.26	5.99	- .13	- 0.16
1941	June	10	2.39	1.38	III	96	1.96	1.96	2.05	2.05	- .09	- .09
1942	Sept.	7	3.89	.22	I	76	1.65		.35		1.30	
		8	3.36		III	96	2.91		2.02		.89	
		9	.78		III	96	.44	5.00	.46	2.83	- .02	2.17
1943	June	5	1.58	.09	I	76	.22	.22	.51	.51	- .29	- .29
1944	April	29	3.63	0	I	76	1.45		1.56		- .11	
		30	2.64		III	96	2.21		2.15		.06	
	May	1	6.37		III	96	5.90		6.92		- 1.02	
		2	1.10		III	96	.73	10.29	.13	10.76	.60	- .47
1945	March	2	.77	.41	I	76	0		.23		- .23	
		3	2.50		III	96	2.07	2.07	2.15	2.38	- .08	- .31
1946	May	12	2.90	1.08	III	96	2.46		2.11		.35	
		13	.95		III	96	.59	3.05	.84	2.95	- .25	.10
1947	March	18	1.74	0	I	76	.29	.29	.85	.85	- .56	- .56
1948	April	25	3.10	.05	I	76	1.08	1.08	1.17	1.17	- .09	- .09
1949	July	4	2.86	.03	I	76	.92	.92	1.07	1.07	- .15	- .15
1950	Feb.	12	1.94	1.08	III	96	1.52	1.52	1.09	1.09	.43	.43
1951	June	16	1.64	1.28	II	89	.74	.74	.19	.19	.55	.55

5. Determine which AMC applies for each rain in column 4, table 10.2. The AMC for the first day of a multiple-day storm is obtained by use of dates in columns 2 and 3 (to get the season), antecedent rainfall in column 5, and figure 4.9. The AMC for succeeding days in a multiple-day storm is similarly obtained but with the previous day's rain (from column 4) added to the antecedent rainfall. The results are shown in column 6. The CN for the AMC are shown in column 7.

6. Estimate the runoff for each day. Enter figure 10.1 with the rainfall in column 4 and the CN in column 7 and estimate the runoff. The results are tabulated in column 8.

7. Add the daily runoffs in a storm period to get the storm total. The totals are shown in column 9. This step completes the example.

Actual runoffs for W-1, taken from reference 2, are given in columns 10 and 11 for comparison with the estimates in columns 8 and 9. Differences between computed and actual runoffs are shown in columns 12 and 13. For some estimates the differences (or estimation errors) are fairly large; the errors may be due to one or more of several causes, of which the most obvious is applying an average land use and treatment to all years and all seasons in a year. The quality of land use and treatment varies (that is, the CN varies from the average) from year to year because of rainfall and temperature excesses or deficiencies and during the seasons of a year because of stages in crop growth as well. In practice the magnitudes of the variations are generally unknown so that the method of this example is usually followed; if they are known, the CN are increased or decreased on the basis of the hydrologic condition as described in the next section. A comparison made later in this chapter illustrates that errors of estimate, even when fairly large, do not adversely affect frequency lines constructed from the estimates as long as the errors are not all of one type.

SEASONAL OR ANNUAL VARIATIONS. The average CN in table 9.1 apply to average crop conditions for a growing season. If seasonal variations in the CN are desired, the stages of growth of the particular crop in the complex indicate how much and when to modify the average CN.

For cultivated crops in a normal growing season the CN at plowing or planting time is the same as the CN for fallow in the same soil group of table 9.1; midway between planting and harvest or cutting times the CN is the average in table 9.1; and at the time of normal peak growth or height (usually before harvest) the CN is:

$$CN_{\text{normal peak growth}} = 2\,(CN_{\text{average}}) - (CN_{\text{fallow}}) \qquad (10.13)$$

Thus, if the average CN is 85 and the fallow CN is 91, the normal peak growth CN is 79. After harvest the CN varies between those for fallow and normal peak growth, depending on the effectiveness of the plant residues as ground cover. In general, if 2/3 of the soil surface is exposed, the fallow CN applies; if 1/3 is exposed, the average CN applies; and if practically none is exposed the normal peak growth CN applies.

For pasture, range, and meadow, the seasonal variation of CN can be estimated by means of tables 8.1 and 8.2; for woods or forest, the Forest Service method in chapter 9 is applicable.

Changes in CN because of above- or below-normal rainfall or temperature occur not only from year to year but also within a year. They are more difficult to evaluate than changes from normal crop growth because detailed soil and crop histories are necessary but seldom available; climate records are a poor substitute even for estimating gross departures from normal. Runoff records from a nearby streamflow station are a better substitute because they provide a means of relating CN to a runoff parameter (for an example see figure 5.6(a)) and approximating the variations of CN.

The CN of table 9.1 do not apply for that portion of the year when snowmelt contributes to runoff. The methods of chapter 11 apply for melt periods. Chapter 12 contains a discussion of snow or freezing in relation to land use and treatment.

VARIATION OF RUNOFF DURING A STORM. The variation of runoff during the progress of a storm is found by the method of the following example. This method is also used for design storms in chapter 21.

Example 10.7.--Estimate the hourly pattern of runoff for a watershed having a CN of 80 and condition AMC-II before a storm of 20 hours' duration, using rainfall amounts recorded at a rain gage.

1. Tabulate the accumulated rainfalls at the accumulated times. Accumulated times are shown in column 1, rainfalls in column 2, of table 10.3

2. Estimate the accumulated runoff at each accumulated time. Use the CN and the rainfalls of column 2 to estimate the runoffs by means of figure 10.1. The runoffs are given in column 3.

3. Compute the increments of runoff. The increments are the differences given in column 4. Plotting these increments shows the pattern of runoff (the plotting is not given).

Table 10.3.--Incremental runoffs for a storm of long duration

Time	Accumulated rainfall	Accumulated runoff	ΔQ
	(inches)	(inches)	(inches)
1:00 a.m.	0	0	
2:00	.15	0	0
3:00	.30	0	0
4:00	.62	0	0
5:00	1.01	.08	.08
6:00	1.27	.18	.10
7:00	1.36	.22	.04
8:00	1.36	.22	0
9:00	1.38	.23	.01
10:00	1.38	.23	0
11:00	1.55	.32	.09
12:00 noon	1.87	.48	.16
1:00 p.m.	2.25	.72	.24
2:00	2.61	.97	.25
3:00	2.66	1.00	.03
4:00	2.68	1.01	.01
5:00	3.22	1.42	.41
6:00	4.17	2.18	.76
7:00	4.82	2.74	.56
8:00	4.93	2.83	.09
9:00	5.00	2.89	.06

RUNOFF FROM URBAN AREAS. Whether a conversion of farmlands to urban area causes larger amounts of storm runoff than before depends on the soil-cover complexes existing before and after the conversion; determination of the "before" and "after" CN is sufficient for a decision. A comparison of runoffs, using real or assumed rainfalls, gives a quantitative answer. Impervious surfaces of an urban area cause runoff when the remainder of the area does not so that the method of example 10.4 is best used. But these surfaces may not contribute runoff in direct ratio to their proportion in the area as the following case illustrates.

Figure 10.3 shows storm rainfall amounts plotted versus runoff amounts for Red Run, a fully urbanized watershed of 36.5 square miles' drainage area, near Royal Oak, Michigan. The data are from "Some Aspects of the Effect of Urban and Suburban Development upon Runoff" by S. W. Wiitla; open-file report, U.S. Geological Survey, Lansing, Michigan; August 1961. This watershed has 25 percent of its area in impervious surfaces and presumably runoff amounts should never be less than those shown by the 25-percent line on the figure. But the data show that the surfaces are only about half effective in generating runoff. The report does not state why this deficiency occurs but does state that "Flood peaks on the urban basin were found to be about three times the magnitude of those for natural basins of comparable size." Determination of the effects of urbanization may therefore require as much use of the methods in chapters 16 and 17 as of those in this chapter.

APPLICATIONS TO RIVER BASINS OR OTHER LARGE AREA. The runoff-estimation method is not restricted to use for small watersheds. It applies equally well to river basins or other large areas providing the geographical variations of storm rainfall and soil-cover complex are taken into account; this is best accomplished by working with hydrologic units (chap. 6) of the basin. After runoff is estimated for each unit the average runoff at any river location is found by the area-runoff weighting method of example 10.4.

INDEXES FOR MULTIPLE REGRESSION ANALYSES. The parameter CN is not a desirable index of watershed characteristics in a multiple regression analysis (chap. 18) because there is generally insufficient variation in the CN to provide a statistically significant result. The parameter S is the preferred index. It is used without change if it is an independent variable in a regression equation with the final form of:

$$Y = a + b X_1 + c X_2 \; \ldots\ldots \qquad (10.14)$$

where Y is the dependent variable; a, b, c, etc. are constants; and the subscripted X's are the independent variables. But if the final form is

$$Y = a X_1^b X_2^c \ldots.. \qquad (10.15)$$

it is necessary to use (S + 1) instead of S to avoid the possibility of division or multiplication by zero. The equation for lag used to develop figure 15-3 uses (S + 1) for this reason; otherwise the graph would give a lag of zero time for an impervious surface (because S is zero when CN is 100) no matter how large an area it might be.

ACCURACY. Major sources of error in the runoff-estimation method are the determinations of rainfall and CN. Chapter 4 provides graphs for estimating the errors in rainfall. There is no comparable means of estimating the errors in CN of ungaged watersheds; only comparisons of estimated and actual runoffs indicate how well estimates of CN are being made. But comparisons for gaged watersheds, though not directly applicable to ungaged watersheds, are useful as guides to judgment in estimating CN and as sources of methodology for reducing estimation errors.

A comparison of storm totals in example 10.6 shows that estimated amounts are fairly close to recorded amounts in 7 out of 12 years, despite the use of a CN for average land use and treatment. On the whole, this is acceptable estimation in view of the limitation on the CN. But the results are better if the storm totals are used as data in a frequency analysis (chap. 18). Figure 10.4(a) shows data from columns 9 and 11, table 10.2, arranged in order of magnitude in their respective groups, and plotted versus their sample percent-chance values. Solid or broken lines connecting the points identify the groups. It is evident from the plotting that one frequency line serves equally well for either group. Thus the estimation errors, though large for some estimates, do not preclude the construction of an adequate frequency relationship. The reason is that the errors are random, being neither all plus or all minus nor all confined to a particular range of magnitudes.

The example of W-1 at Waco demonstrates that estimation errors should be kept random. One way of accomplishing this is to apply the CN for AMC-II to all storms in a series. A second example illustrates this.

Storm runoffs and rainfalls for Amicalola creek, Georgia, are given in columns 5 and 6 of figure 5.5. The CN is 65 for AMC-II, as determined in example 5.4. This CN and the rainfalls give the following estimates of runoff (actual runoffs are shown for comparison):

Year	Runoff (in.)		Year	Runoff (in.)	
	Estimated	Actual		Estimated	Actual
1940	1.64	0.81	1947	1.06	1.59
1941	2.15	1.40	1948	2.13	1.36
1942	1.81	1.74	1949	2.06	1.85
1943	1.22	1.65	1950	.89	1.15
1944	.91	1.16	1951	1.46	1.33
1945	.12	.36	1952	.93	2.01
1946	1.92	2.33			

In a plotting of estimated versus actual runoff the scatter of points indicates a moderately low degree of correlation, but the scatter also indicates that the errors are randomly distributed, which means that a reasonably good result on probability paper can be expected. Figure 10.4(b) substantiates this: again a single frequency line will do for either group. The curvature of the plottings signifies only that 13 years of record on this watershed are insufficient for an adequate frequency line (chap. 18); discrepancies in the lower half of the plotting come from this insufficiency.

In practice the CN for an ungaged watershed cannot be estimated by means of runoff data, as the CN for Amicalola Creek was, but it can be estimated from watershed data at least as well as that for W-1 at Waco. It will take correct identification of soil-cover complexes, especially if there are few complexes in a watershed or they differ little from each other or one of them dominates the area. But if there are many complexes of about equal area and in a wide range of CN, it is likely that misjudgment of several will not adversely affect the estimate of the average CN. Using complexes that are properly identified and rainfall data that are adequate, runoff estimates are made accurately enough for practical purposes.

* * * *

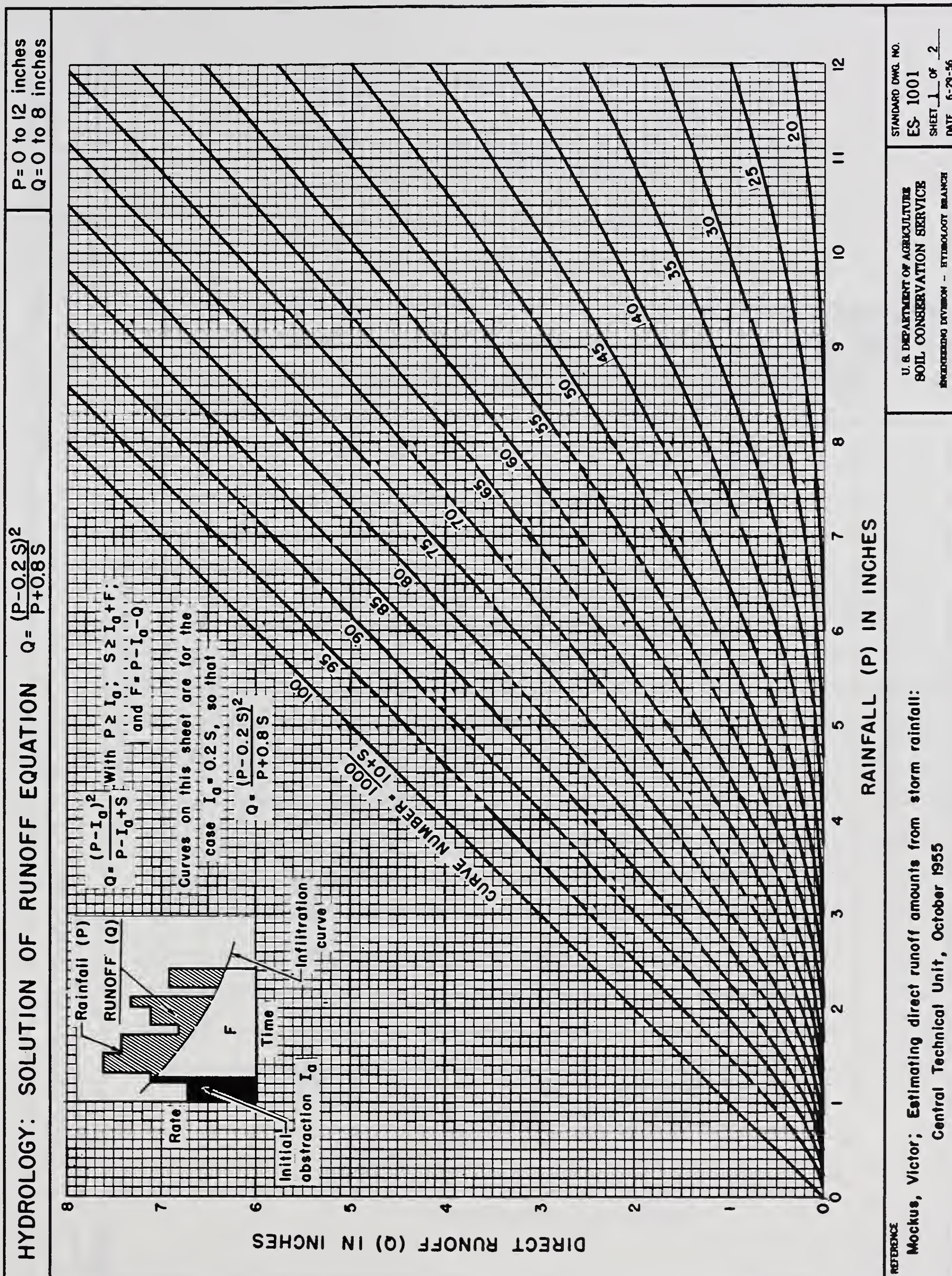
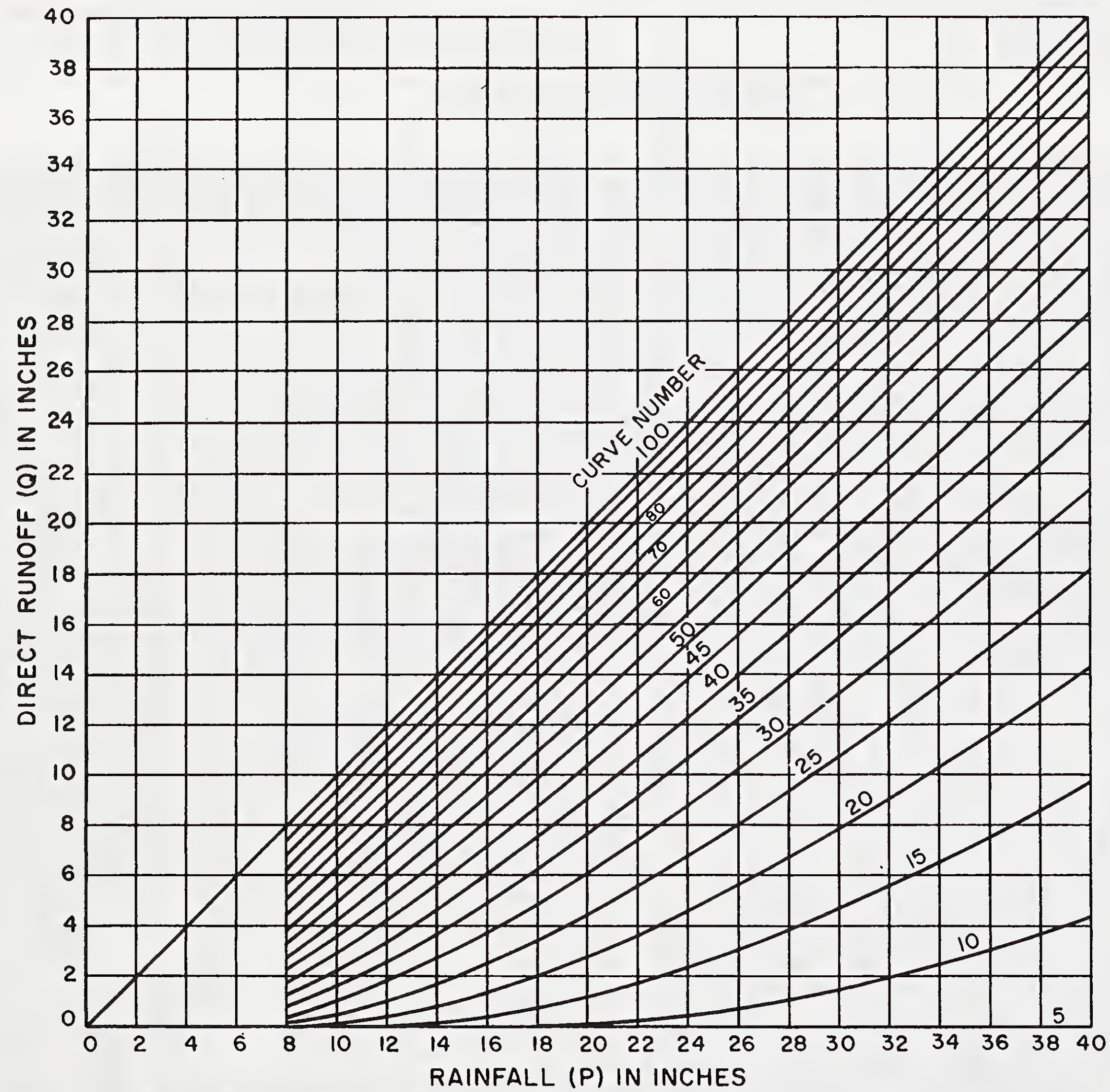


Figure – 10.1 (1 of 2)

HYDROLOGY: SOLUTION OF RUNOFF EQUATION $Q=\frac{(P-0.2S)^2}{(P+0.8S)}$

P=8 to 40 inches
Q=0. to 40 inches



REFERENCE
Mockus, Victor; Estimoting direct runoff omounts from storm roinfoll: Centrol Technicol Unit, October 1955.

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ENGINEERING DIVISION – HYDROLOGY BRANCH

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Figure 10.1 (2 of 2)

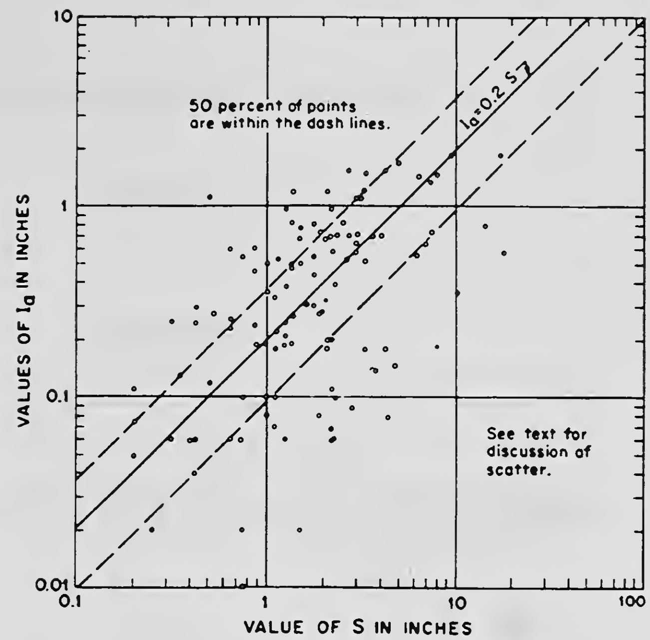

Figure 10.2.--Relationship of I_a and S. Plotted points are derived from experimental watershed data.

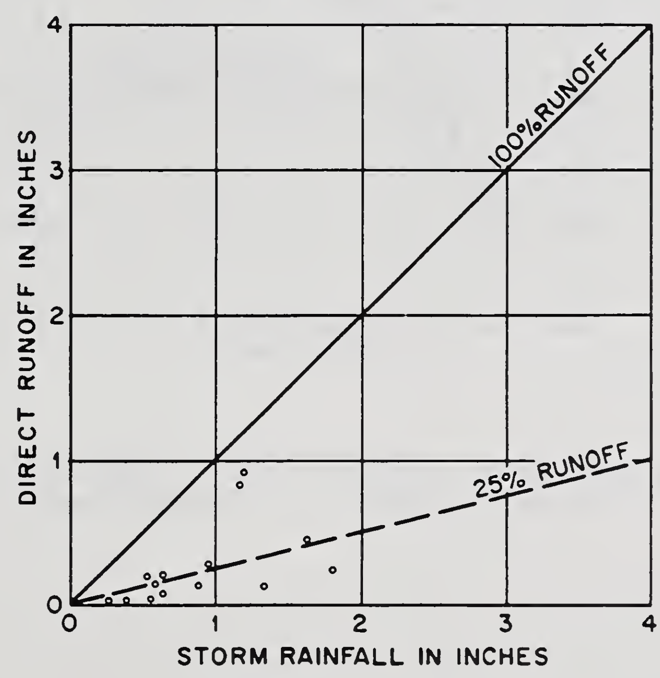

Figure 10.3.--Expected minimum runoff (dashed line) and actual runoff (plotted points) for an urbanized watershed.

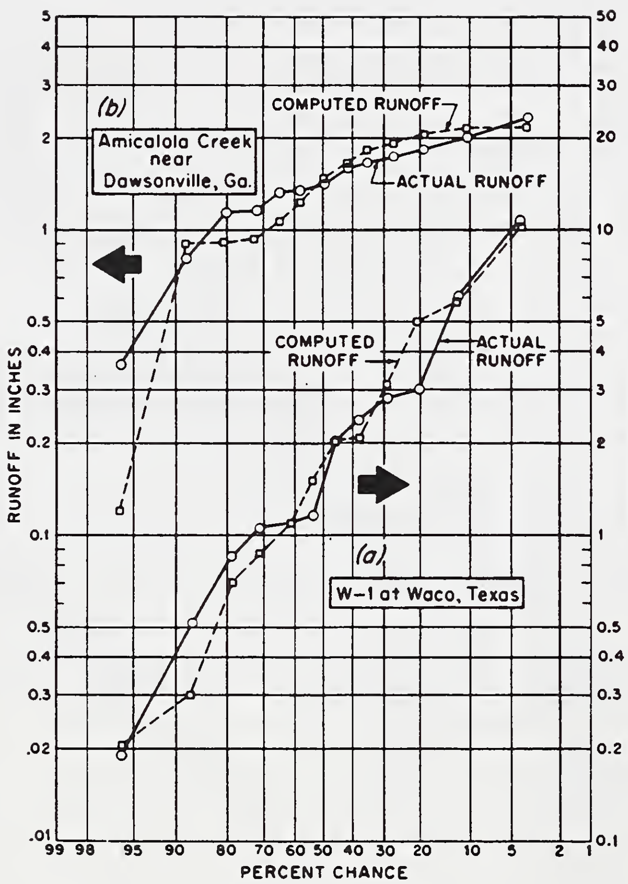

Figure 10.4.--Comparisons of computed with actual runoff on a frequency basis.

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HYDROLOGY

CHAPTER 11. ESTIMATION OF DIRECT RUNOFF FROM SNOWMELT

by

Victor Mockus
Hydraulic Engineer

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CHAPTER 11. ESTIMATION OF DIRECT RUNOFF FROM SNOWMELT

This chapter gives methods for estimating snowmelt runoff volumes for flood damage evaluations. Methods of snowmelt forecasting, for irrigation and similar purposes, are described in the Snow Survey Handbook of the Service.

Details of the thermodynamics of snowmelt are omitted from this chapter because of their limited value in the methods presented here. Some standard references are:

Clyde, George D. - Snow-melting characteristics. Technical Bulletin 231, August 1931. Utah Agricultural Experiment Station, Logan, Utah.

Light, Phillip - Analysis of high rates of snowmelting. Pages 195-205, Transactions of the American Geophysical Union, 1941.

Wilson, W. T. An outline of the thermodynamics of snowmelt. Pages 182-195, Transactions of the American Geophysical Union, 1941.

Significance of Snowmelt Floods

Bankfull capacities in csm are normally greater for small watersheds than for large ones. Since snowmelt rates are relatively low in csm there may be flooding on large watersheds when streams on small watersheds are flowing less than bankfull.

The hydrologist acquainted with an area will know the relative importance of snowmelt as a source of flooding in that are. In doubtful cases the data normally gathered by interview for an historical flood series will usually define the character of flood flows. In other instances, the runoff records will show how important snowmelt flooding is. It is seldom necessary to make detailed hydrologic investigations into the matter.

Methods of Estimation

Regional analysis
This method is one of the most useful for snowmelt floods. See Chapter 2 for details of the method.

Degree-day method, ungaged watersheds
This method is widely used because of its adaptability to usual data conditions. Similar methods going into more detail are available but seldom applicable because of lack of required data.

The degree-day method uses the equation:

$$M = K D \tag{11-1}$$

where M = the watershed snowmelt in inches per day.

K = a constant that varies with watershed and climatic conditions.

D = the number of degree-days for a given day.

A degree-day is a day with an average temperature one degree above 32° F. Maximum and minimum temperatures, as found in "Climatological Data," are averaged to get the daily average temperature. A day with an average of 40° F. gives eight degree-days; with an average of 51° F., nineteen degree days. The general form of the method is given below. A working arrangement of the data is shown on table 11-1. In most cases the table can be condensed. The steps in the method are:

1. Using precipitation stations or snow survey data, show either (a) the total available water equivalent at the beginning of the melt season (table 11-1) or (b) the precipitation and the water equivalent by days (table 11-2). The first procedure is used where there is generally only one melt period per year; the second, where melt periods occur intermittently through the winter and spring. Water equivalent is the depth of water, in inches, that results from melting a given depth of snow, and it is dependent on both depth and density of snow. Snow surveys give field determinations of water equivalents. Where such surveys are not made, it is customary to use one-tenth of the snow depth as the depth of water equivalent.

2. For temperature stations in the watershed, tabulate average temperatures for the melt periods. (Note: maximum and minimum values as given in "Climatological Data" can be averaged mentally to avoid tabulation of averages below 33° F.)

Table 11-1. Estimation of snowmelt by degree-day method. One melt period

Dates	Watershed average temperature °F. 1/	Degree-days	Estimated snowmelt 2/	Total available water equivalent
			Inches	Inches
April 5	32	0	0	4.50
6	35	3	.18	4.32
7	34	2	.12	4.20
8	36	4	.24	3.96
9	48	16	.96	3.00
10	43	11	.66	2.34
etc.	etc.	etc.	etc.	etc.

1/ Average of two stations; adjusted for altitude.

2/ Using K = 0.06 in equation 11-1.

Table 11-2. Estimation of snowmelt by degree-day method. Intermittent melt period.

Dates	Precipitation	Water equivalent 1/	Degree-days	Snowmelt: Potential 2/	Snowmelt: Estimated	Remaining water equivalent
	Inches	Inches		Inches	Inches	Inches
Nov 3	0.85	0.08				0.08
Nov 4-18						.08
Nov 19			5	0.30	0.08	0
Nov 20-29						0
Nov 30	3.80	.38				.38
Dec 1-24						.38
Dec 25	4.15	.42				.80
Dec 26-Jan 18						.80
Jan 19	.52	.05				.85
Jan 20-Feb 2						.85
Feb 3-20	6.92	.69				1.54
Feb 21-Mar 14						1.54
Mar 15	14.24	1.42				2.96
Mar 16-28						2.96
Mar 29			3	.18	.18	2.78
Mar 30			11	.66	.66	2.12
Mar 31			22	1.32	1.32	.80
Apr 1-9						.80
Apr 10			7	.42	.42	.38
Apr 11			32	1.92	.38	0

1/ One-tenth of snow depth.
2/ Using K = 0.06 in equation 11-1.

3. Adjust the average temperatures to the average watershed elevation, using the method given below in Adjustment of temperatures for altitude. This step is omitted when elevation data are crude or otherwise unreliable.

4. Compute the watershed average daily temperatures by averaging the station averages (adjusted for altitude, if desirable).

5. Subtract 32° F. from each watershed average daily temperature to get the degree-days per day.

6. Use equation 11-1 to get an estimate of the potential snowmelt for each day. See K factors below for selection of K.

7. Where the daily potential is not greater than the water equivalent remaining on the watershed, it is shown as an estimate of snowmelt.

Once the estimates of snowmelt are obtained, they are used to obtain hydrographs as described in Chapter 16.

Some hydrologists suggest that the effects of infiltration be subtracted from the estimated snowmelt. However, the K factors as generally developed already include the effects of infiltration. The effects of measures such as contour furrows are obtained as described in Chapter 12. The effects of reservoirs, levees, etc. are obtained as usual.

Refinements in the degree-day method are best made by first improving the accuracy of determinations of snow depth and areal distribution on the watershed. When these are known within small limits of error, then water equivalents should be refined, since the 1/10 ratio is a rough approximation. Refinements in K factors should come last.

Degree-day method, gaged watershed

The degree-day method has a very limited use, if any at all, for flood evaluations on gaged watersheds. When gaging station data are available, those data should be used to estimate flood peaks and volumes on other portions of the watershed.

Adjustment of temperatures for altitude

In general, air temperatures decrease about 3° to 5° for every 1,000 feet of rise in altitude. Other factors influence this "lapse rate," so that refinements are not justified, and an average decrease of 4° F. per 1,000 feet rise should be used.

<u>Example 11-1</u>--A watershed with an average elevation of 4,600 feet had temperature station readings of 38° F. at a 5600-foot elevation, and 48° F. at a 3000-foot elevation. The average temperature for the watershed is then:

$$(38) - \frac{4}{1000}(4600 - 5600) = 42.0$$
$$(48) - \frac{4}{1000}(4600 - 3000) = 41.6$$

Sum: 83.6

Average: 41.8

Round off to: 42

While further refinements, such as weighting, can be made, they are seldom justified.

<u>K factors</u>
The constant K in equation 11-1 is known to vary not only from watershed to watershed, but also from day to day on a given watershed. It is seldom possible to do more than make a broad estimate of K. An average value of 0.06 can be used. The following table may be of assistance in special cases:

Table 11-3. K factors

Condition	K
Extremely low runoff potential	0.02
Average heavily-forested areas; north-facing slopes of open country	.04 - .06 1/
Average	.06
South-facing slopes of forested areas; average open country	.06 - .08 1/
Extremely high runoff potential	.30

1/ Recommended by A. L. Sharp.

<u>Concordant flow method</u>
The method of Chapter 2 can be simplified to estimate both peaks and volumes of snowmelt runoff, when at least one streamflow record is available. The method is very similar to the <u>Regional analysis</u> method mentioned above and in Chapter 2.

The volume of snowmelt for an ungaged subwatershed is the same as that for the gaged watershed, assuming equal coverage of snow over both areas. Where it is possible to estimate the amounts or degrees of snow coverage, the snowmelt volumes in inches may be taken as directly proportional to snow depth or degree of coverage. For example, if there is a 3.2" snowmelt runoff from a gaged watershed of 82 square miles with 76 percent of the watershed having snow cover, then a subwatershed of 12 square miles and 100 percent snow cover will have an estimated runoff of:

$$3.2 \frac{(100)}{(76)} = 4.2 \text{ inches.}$$

Note that area in square miles is not used in the computation unless acre-feet are needed. If instead of the percents the gaged watershed is known to have an average of 16.2 inches of snow-depth and the ungaged subwatershed 20.4 inches, then the runoff for the subwatershed is:

$$3.2 \frac{(20.4)}{(16.2)} = 4.0 \text{ inches.}$$

Other factors can be brought in, but here again refinement is not justified.

Peaks of snowmelt runoff can be obtained as described in Chapter 16.

<u>Other methods</u>
Where intensive study has been or can be made of a watershed, more detailed and more accurate methods of estimating snowmelt runoff can be used.

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CHAPTER 12. HYDROLOGIC EFFECTS OF LAND USE AND TREATMENT

by

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Hydraulic Engineer

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CHAPTER 12. HYDROLOGIC EFFECTS OF LAND USE AND TREATMENT

The effects that are discussed here are (a) changes in volumes of direct runoff and (b) changes in lag, which affect peak rates of direct runoff.

Volume Effects

Land use and treatment measures reduce the volume of direct runoff during individual storms by either (1) increasing infiltration rates, or (2) increasing surface storage, or both. Other factors influencing runoff volume are usually of minor importance. Interception increases, for instance, are appreciable only under certain climatic and vegetative conditions and generally need not be considered in Service watershed studies.

The unit hydrograph principle states that with other things constant, the peak rate of flow varies directly with the volume of flow. This principle is the basis for proportionate reductions in peaks when volumes are reduced (see Chapter 16). Figure 12-1 shows a typical peak vs. volume relation. The straight line is drawn so that some points are on the line, if possible, with half of the remaining points on one side of the line and the other half on the other side. Drawing a curve is not justified, since other important relations must be accounted for (see Chapter 16) if greater accuracy is required. The figure shows that a 30 percent reduction in volume gives a 30 percent reduction in the peak rate, and so on.

Table 12-1 shows the principal effects of land use and treatment measures on direct runoff. The degree of effect of any single measure generally depends on the quantity that can be installed. Contour furrows, however, can be made to have a small or large effect by changing the dimensions of the furrows. The effect of a land use change depends on the change in cover. A change from spring oats to spring wheat would ordinarily be hardly noticeable, while a change from oats to a permanent meadow could have a large effect. Graded terraces with grass outlets to some extent will increase both over-all infiltration and over-all storage. These effects are also confused with a lag effect. It should be noted that lime and fertilizers, by increasing plant or root density, can indirectly reduce direct runoff volumes.

Table 12-1. Principal effects of land use and treatment measures on direct runoff.

Measure	Reduction in direct runoff volume is due to: Increasing infiltration rates 1/	Reduction in direct runoff volume is due to: Increasing surface storage
1. Land use change that increases plant or root density. 2/	X	
2. Increasing mulch or litter	X	
3. Contouring		X
4. Contour furrowing		X
5. Level terracing		X
6. Graded terracing		X

1/ Assuming soils not frozen.

2/ Example: Row crop to grass for hay. Poor pasture to good pasture.

Lag Effects

Lag, as used here, means the delay between the production of direct runoff on upland areas and its appearance at a given cross section in a stream channel. Another discussion of lag is given in Chapter 15.

Land use and treatment measures can produce lag effects by (1) increasing infiltration (reducing surface runoff) and causing the increased infiltration to appear some time later as subsurface flow, or (2) by causing a delay in the arrival of surface runoff by increasing the distance or reducing the velocity of flow.

Either effect is best studied by the methods of Chapters 15 and 16. Table 12-2 shows the relative effects of land use and treatment measures on the two types of lag. The subdivisions of small and large watersheds do not depend solely on size in square miles. The methods of Chapters 15 and 16 are necessary in quantitative studies of lag.

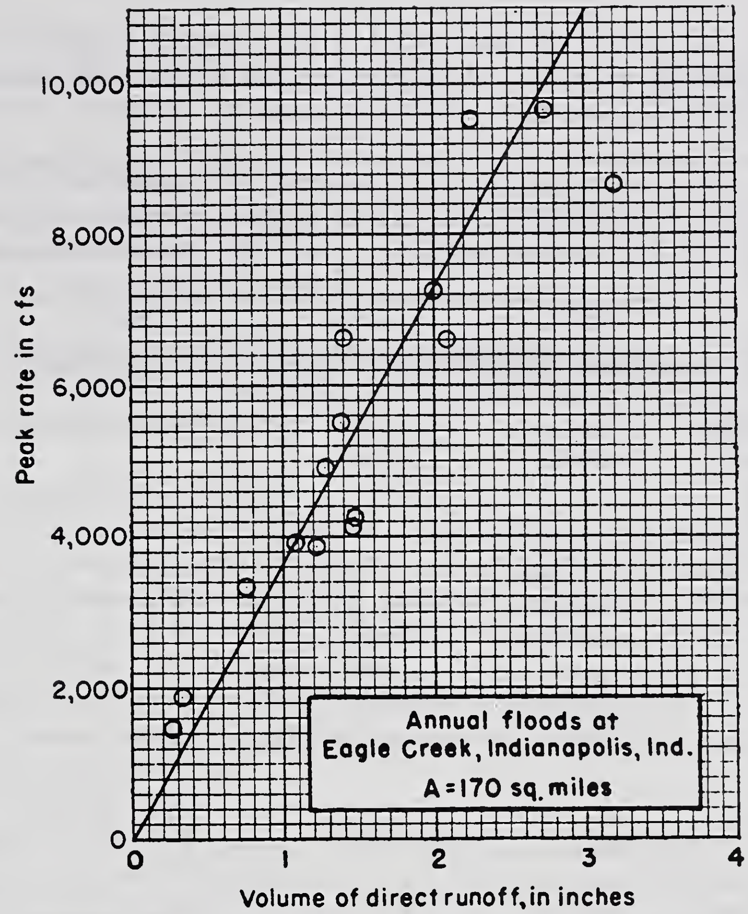

Figure 12-1 Typical peak-volume relationship.

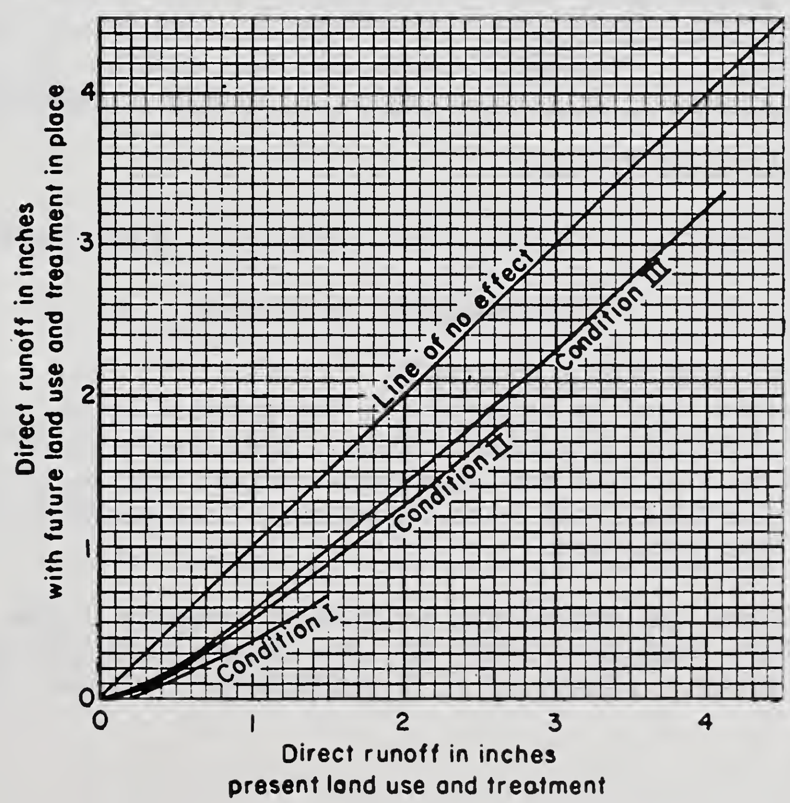

Figure 12-2 Volume effects of land use and treatment.

Table 12-2. Relative effects of land use and treatment measures on types of lag.

Measure	Effect on subsurface flow 1/		Effect of increasing surface flow distance or decreasing velocity	
	Small watersheds	Large watersheds	Small watersheds	Large watersheds
1. Land use changes that increase plant or root density. 2/	Can be large.	Can be large.	Not usually considered.	
2. Increasing mulch or litter.	Can be large.	Can be large.	Not usually considered	
3. Contouring.	Can be large	Usually negligible.	Can be large.	Negligible.
4. Contour furrowing.	Can be large.	Can be large.	Not usually considered.	
5. Level terracing.	Can be large.	Can be large.	Not usually considered.	
6. Graded terracing.	Usually negligible.	Usually negligible.	Can be large.	Negligible.

1/ Assuming soils not frozen.

2/ Examples: Row crop to grass; poor pasture to good pasture.

Determination of Effects

Determination of effects on volume

The same procedure used in determining the present hydrologic conditions of a watershed is used to estimate future hydrologic conditions. The future effects of land use and treatment changes can be estimated with relatively little additional work. Assuming that present conditions have been studied, the steps are:

1. Determine the hydrologic soil-cover complex number, antecedent moisture condition (AMC) II, for future land use and treatment conditions. (Chapters 7, 8 and 9.)

2. Obtain complex numbers for AMC I and III (table 10-1).

3. Prepare a working table similar to table 12-3.

4. Plot the corresponding present and future values as shown on figure 12-2. For example, plot 0.23 vs. 0.02; 0.60 vs. 0.18; and 1.10 vs. 0.43, and draw in the curve for AMC I. Similarly for the other conditions.

5. Enter figure 12-2 with the present volume and condition for a storm or flood in the evaluation series and find future volume on the appropriate curve.

Determination of effects on lag

Increased infiltration appearing some time later as subsurface flow is seldom easy to evaluate quantitatively. Fortunately, however, in most flood prevention surveys the changes in the hydrograph due to this lag effect can generally be neglected. Where it cannot, special studies are needed to determine the source areas (which may vary with infiltrated volumes) and watershed retention. The techniques for these special studies have not been fully developed, however, and the results are likely to be controversial.

Table 12-3. Sample working table. Estimation of effects of future land use and treatment on direct runoff volumes.

Selected values of "P"	Direct runoff, in inches, for selected values of "P", from figure 10-1					
	AMC* I		AMC* II		AMC* III	
	Present	Future	Present	Future	Present	Future
0.5	0	0	0	0	0.08	0
1	0	0	.02	0	.35	.12
2	0	0	.38	.11	1.15	.70
3	.23	.02	.97	.50	2.05	1.45
4	.60	.18	1.68	1.03	3.00	2.30
5	1.10	.43	2.46	1.65	3.95	3.20
Curve numbers:	57	45	75	65	91	83

*AMC is antecedent moisture condition.

Quite often this first type of lag can be assumed to take place in the manner of the second type of lag, and the technique given below can be used to estimate expected changes in hydrograph quantities.

The effect of causing a delay in the arrival of surface runoff by increasing the distance of flow is easily computed when it must be considered. Figure 12-3 shows hydrographs for adjacent treated and untreated watersheds. (For additional data see "Runoff from conservation and non-conservation watersheds" by J. A. Allis, Agricultural Engineering, Vol. 34, No. 11, Nov. 1953.) Two effects are evident. Some of the reduction in peak rate is due to the lesser amount of runoff from the treated watershed. Given the data as shown, the expected peak for the treated watershed would be:

$$1.74 \frac{(1.35)}{(1.68)} = 1.40 \text{ in./hr., since } \frac{q_1}{Q_1} = \frac{q_2}{Q_2} \text{ when runoff is}$$

uniformly (or nearly so) distributed on each watershed, but the actual value for W-5 is 0.87 in./hr. The difference is due primarily to a lag caused by graded terraces and open-end level terraces (which tend to grade).

Following the methods of Chapters 15 and 16, the additional lag can be computed from data on figure 12-3. The time to peak (T_p) for W-3 is about 0.72 hour, and for W-5, about 1.05 hour. The increase in lag (since storm D is essentially identical for both hydrographs) is:

$$1.05 - 0.72 = 0.33 \text{ hour}$$

Since T_p consists of storm duration and time of concentration (see Chapter 16), the changes in either (or both) factors can be studied in a graph like that of figure 12-4. The graph shows that, for this case, the second type of lag effect becomes relatively insignificant at about $T_p = 5$ hours.

Ordinarily, in practice, the second type of lag effect is neglected. The technique given above can be used when the second type must be evaluated and, quite often, for evaluations of the first type of lag effect. The altered hydrographs can be reproduced by the methods of Chapter 16.

Determination of effects on snowmelt runoff

The effects of land treatment on snowmelt runoff may vary considerably from the effects on runoff from rainfall. The principal changes in effects are due partly to changes in the measures themselves, and partly to frost action.

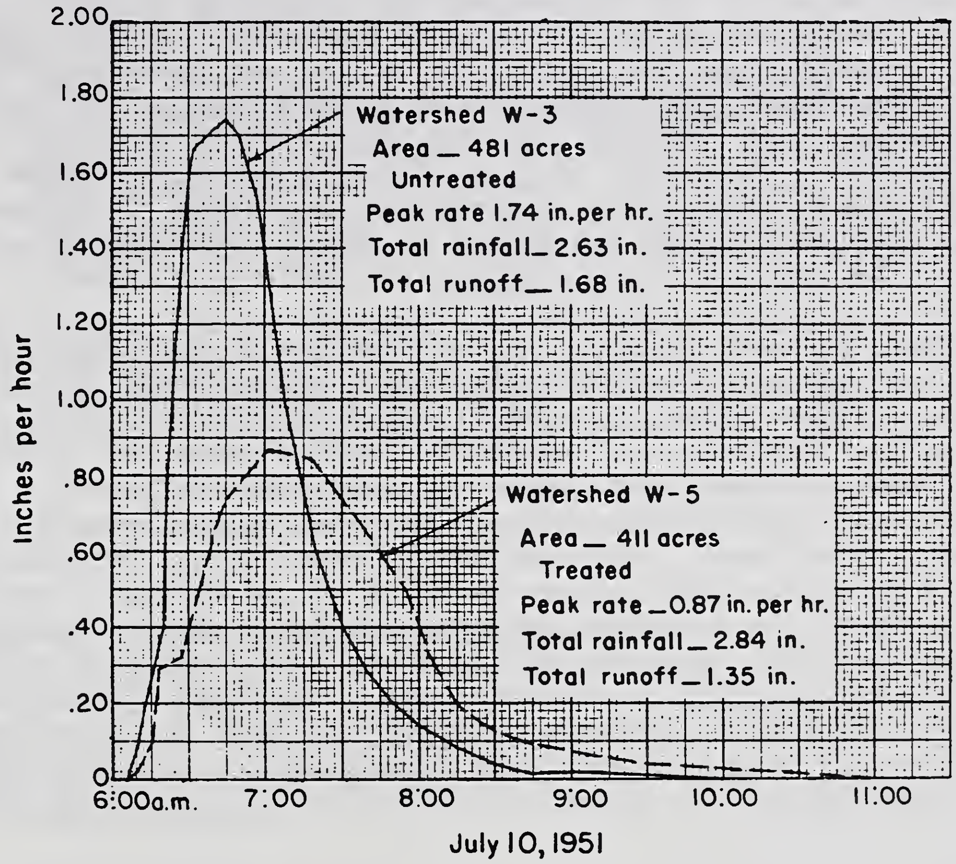

Figure 12-3 Effects of land use and treatment on lag.

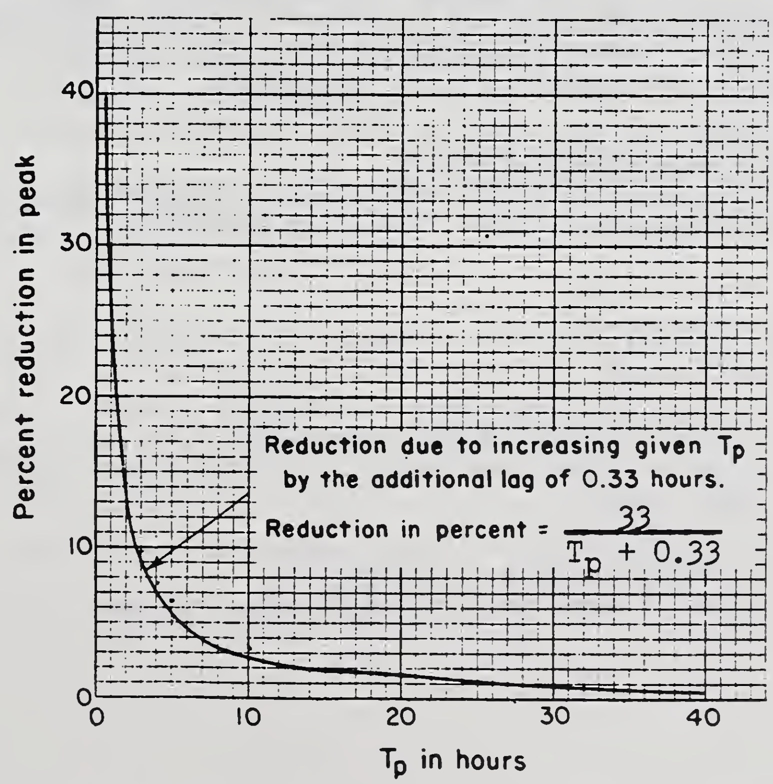

Figure 12-4 Percent peak reduction by increasing lag 0.33 hours and the corresponding increase in T_p.

By the time of arrival of the snow season, cultivation and weathering have usually eliminated the mechanical distinction between straight row and contour farming on cultivated lands. Other effects of contouring are usually small enough to be overshadowed by variations in areal distribution of precipitation and are usually neglected. Graded terracing effects would be confined to the second type of lag and determined by the method shown. Closed-end level terraces and contour furrows are usually dependent on storage, not infiltration, for their effect, which is therefore calculable. The effect of land use or cover on cultivated land and pastures is small enough to be obscured by the effects of topography, fences, roads, and nearby trees and shrubs on the distribution of snow on the ground. The effect of crop rotation is similarly obscured.

In order for land treatment measures to be effective through the snow season, they must either (1) maintain high infiltration rates on soils that have a large water storage potential; or (2) maintain surface storage; but seldom both at once. High infiltration rates are maintained by vegetation that provides heavy litter or large depths of humus. Ordinary practices on cultivated lands and pastures seldom provide sufficient residues and such areas need not be considered. Permanent meadows usually provide enough litter and humus to prevent mild frost action, but not enough to be effective against heavy freezes. Commercial forest and woodland, with the exception of areas like swamps and spruce flats, are effective maintainers of infiltration, and when located on a soil with sufficient internal storage capacity, are very effective in reducing flood runoff from snowmelt. The Forest Service procedure given in Chapter 9 (see figure 9-1) covers the evaluation of commercial forest and woodland.

Surface storage in closed-end level terraces, and in contour furrows, may be effective in reducing snowmelt runoff as described below. Generally, on field-size watersheds, the storage has to be quite large in order to control the additional volumes of snowmelt from snow drifting from adjacent smooth fields and caught by the earthwork.

Determination of surface storage effects

Storage in closed-end level terraces and contour furrows can be evaluated on a watershed or subwatershed basis using the equation:

$$Q_s = \frac{A_s (Q_o - S_s) + A_o Q_o}{A_s + A_o} \qquad (12\text{-}1)$$

where Q_s = runoff in inches with storage in effect
A_s = square miles of area draining into storage and including storage pond area
S_s = storage in inches
Q_o = runoff in inches with no storage
A_o = square miles of area not draining into storage

When S_s exceeds Q_o, then only the storage equal to Q_o is effective. For example, if $S_s = 3.0$ inches and $Q_o = 1.2$ inches, then 1.8 inches of storage have not been used, and the effective storage is 1.2 inches, i.e., when S Q_o, use A_s $(Q_o - S_s) = 0$.

(<u>Note:</u> Equation 12-1 and subsequent equations 12-2, 12-4, 12-5a, and 12-5b, are for use when runoff and storage volumes are distributed uniformly (or nearly so) on a watershed. When the distribution is not uniform, the watershed is divided into subwatersheds on which the distribution may be considered uniform. See remarks accompanying equations 12-5a and 12-5b.)

Infiltration in the storage area, including that due to increased head, is generally assumed to offset storm rainfall on the storage pond area. When this infiltration is significantly large or small, it can be accounted for on a volumetric basis by changing equation 12-1 to read:

$$Q_s = \frac{A_p (P - F) + (A_s - A_p)(Q_o - S_s) + A_o Q_o}{A_s + A_o} \qquad (12\text{-}2)$$

where A_p is the average pond surface area in square miles; P is the storm rainfall, in inches; and F is the total infiltration, in inches, on the area occupied by the pond. If P is less than F, use (P - F) equal to zero. When other data are lacking, and the average depth of the pond is less than about 3 feet, F may be approximated using the following equation:

$$F = D\, f_c\, (1.5\, h + 1) \qquad (12\text{-}3)$$

where F = total infiltration in inches on the pond area
D = storm duration in hours for equation 12-2, or snowmelt duration in hours for equation 12-4
f_c = minimum infiltration rate in inches per hour
h = average depth of pond in feet during time D

Acres or square feet may be used instead of square miles in equations 12-1 and 12-2, but whichever unit is chosen must be used for all the areas in a particular computation.

The effect of storage on snowmelt runoff is generally computed by equation 12-1 since the increase in infiltration due to head in the pond area is usually negligible because of the temperature. When this infiltration is important, equation 12-2 becomes:

$$Q_s = \frac{(A_s - A_p)(Q_o - S_s) + A_o\, Q_o - A_p(Q_o - F)}{A_s + A_o} \qquad (12\text{-}4)$$

unless there is rainfall on the pond surface during the melt period, in which case equation 12-2 is used. The effect of the earthwork in increasing the average depth of snow on an area (by catching drifting snow) is important only on very small areas, and is usually ignored.

According to unit hydrograph theory, the effect of surface storage on peak rate of flow is proportional to the effect on volume of flow when both the storage and runoff are about equally distributed over the watershed:

$$\frac{q_s}{q_o} = \frac{Q_s}{Q_o} \tag{12-5a}$$

or

$$q_s = q_o \frac{Q_s}{Q_o} \tag{12-5b}$$

where q_s is the reduced peak, q_o is the original peak, and the other symbols are as before. Equation 12-5b is adequate for many watersheds. However, when the distribution of Q_o and S_s is not sufficiently uniform, or when a watershed has a complex drainage pattern, or is unusually shaped, or has channel improvements, it is necessary to determine the storage effects on a subwatershed basis, prepare hydrographs on a subwatershed basis, and route floods, in order to determine q_s. It is usually necessary to follow this routing procedure for large watersheds, since the distribution of Q_o and S_s is nearly always nonuniform on large watersheds.

NATIONAL ENGINEERING HANDBOOK

SECTION 4

HYDROLOGY

CHAPTER 13. STAGE-INUNDATION RELATIONSHIPS

by

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NATIONAL ENGINEERING HANDBOOK

SECTION 4

HYDROLOGY

CHAPTER 13. STAGE-INUNDATION RELATIONSHIPS

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CHAPTER 13. STAGE-INUNDATION RELATIONSHIPS

The economist requires data or curves showing the relation between the area inundated and (1) stage, (2) discharge, (3) flood volume, or (4) frequency. The hydrologist generally provides information on these relations, using data obtained in field surveys by both survey engineers and economists. The party leader chooses one of the above relations according to the problem at hand. The hydrologist, therefore, should learn the specific needs of the economist before determining area-inundated relations.

Stage Versus Area Inundated Methods

Simple cases

This method relates the flooded acres in a stream reach to the stage at either end (or middle) of the reach, usually the downstream end, except when the concordant flow method is used (see Chapter 2). As given to the economist, the stage-inundation relation shows the number of acres flooded at depths selected by the economist.

The simplest case occurs when one cross section is used to represent conditions in a reach. Table 13-1 shows a typical computation of a stage versus total-area-inundated relation for this case.

The acres inundated at selected depths of flooding are computed as shown in table 13-2. Figure 13-1a shows the results as generally given to the economist. Note that the curves of acres flooded at given depth increments can also be obtained directly from the "total acres" curve by use of an engineer's scale.

Complex cases

The computation of this relation becomes more laborious when more than one cross section per reach is used, the labor increasing about in proportion to the number of cross sections to be averaged. The computation also becomes complex if a variable length of reach is used, but this procedure is seldom followed for determining acres flooded. The number of acres flooded at various depths is sometimes obtained by planimetering the areas between flow lines plotted on a map of the floodplain.

Table 13-1. Sample computation of stage versus area inundated, for a simple case using one representative cross section in the reach.

Stage	Cross section top width	Width minus channel width	Inundated area in reach	Remarks
Feet	Feet	Feet	Acres	
4	24	0	0	Bankfull stage
6	92	68	13.5	
8	367	343	68.2	
10	608	584	116.0	
12	786	762	151.2	
14	872	848	168.2	

Column 4 is computed using Column 3 and the valley length of the reach. In this case the reach is 8640 feet long. To get acres, the formula is:

$$\frac{8640}{43560} \text{ (Col. 3)} = 0.1984 \text{ (Col. 3)} = \text{(Col. 4)}$$

Slide rule computations.

Table 13-2. Sample computation of stage versus area inundated at selected depths of flooding.

		Acres inundated at given depths			
Stage (Feet)	Total area inundated (Acres)	0-2 (Feet)	2-4 (Feet)	4-6 (Feet)	Over 6 1/ (Feet)
4	0	0	0	0	0
6	13.5	13.5	0	0	0
8	68.2	54.7	13.5	0	0
10	116.0	47.8	54.7	13.5	0
12	151.2	35.2	47.8	54.7	13.5
14	168.2	17.0	35.2	47.8	68.2

Values in columns 3, 4, 5, and 6 can also be obtained graphically. See figure 13-1a, and text.

1/ Values in last column are those of Column 2 shifted downward three lines.

When two cross sections per reach are used, and the drainage areas at the sections are not significantly different in size, the sections may be averaged as shown in table 13-3. Determination of acres flooded for given depth increments follows the procedure of table 13-2. When the two cross sections have significantly different sizes of drainage areas, the sections may be averaged as shown in table 13-4, with the procedure of table 13-2 used to get flooding by depth increments. In this case, the inundated acreage has been related to the foot of the reach. The footnote on table 13-3 tells how the acreage may be related to the middle of the reach for that method. The method given in table 13-4 is probably at its best when acreage is related to the foot of the reach, as shown.

In table 13-4, column 3, the corresponding discharges at the upstream cross section have been proportioned using the ratio of the bankfull discharges. This method is applicable when the channels are not excessively eroded or silted. The method of taking the same discharge in csm is sometimes used, but this method ignores the fact that the upstream bankfull discharge in csm is normally greater (for natural channels in noncohesive materials and in an equilibrium condition or nearly so) than the downstream bankfull discharge in csm. In these cases the exact discharges that should be used are those of the same

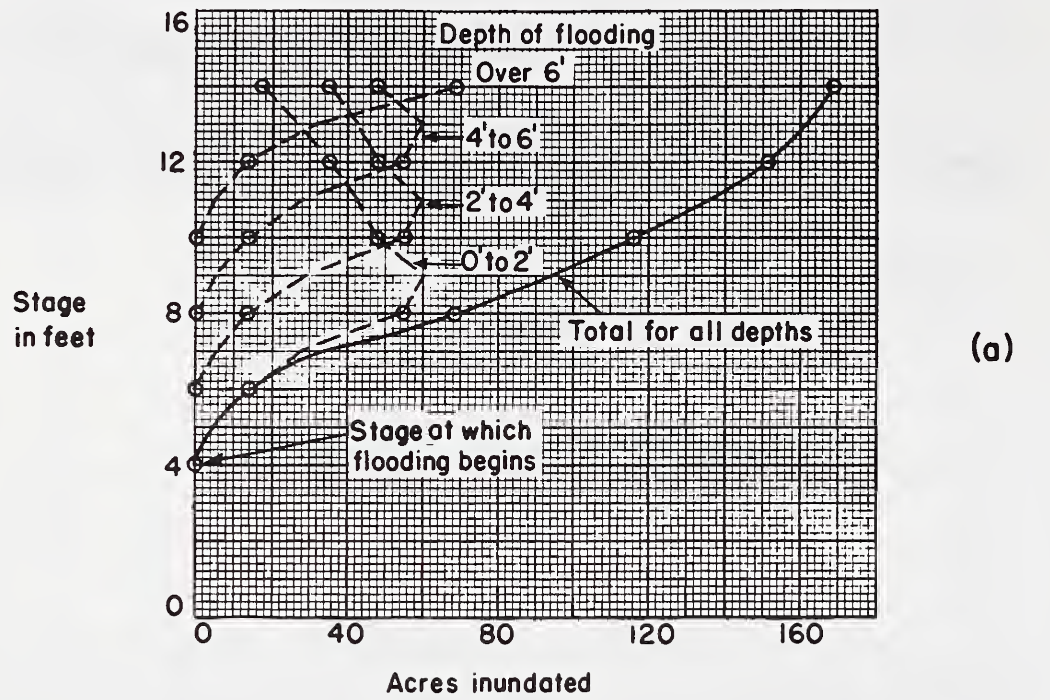

Figure 13-1(a) Area flooded at given depth of flooding increments.

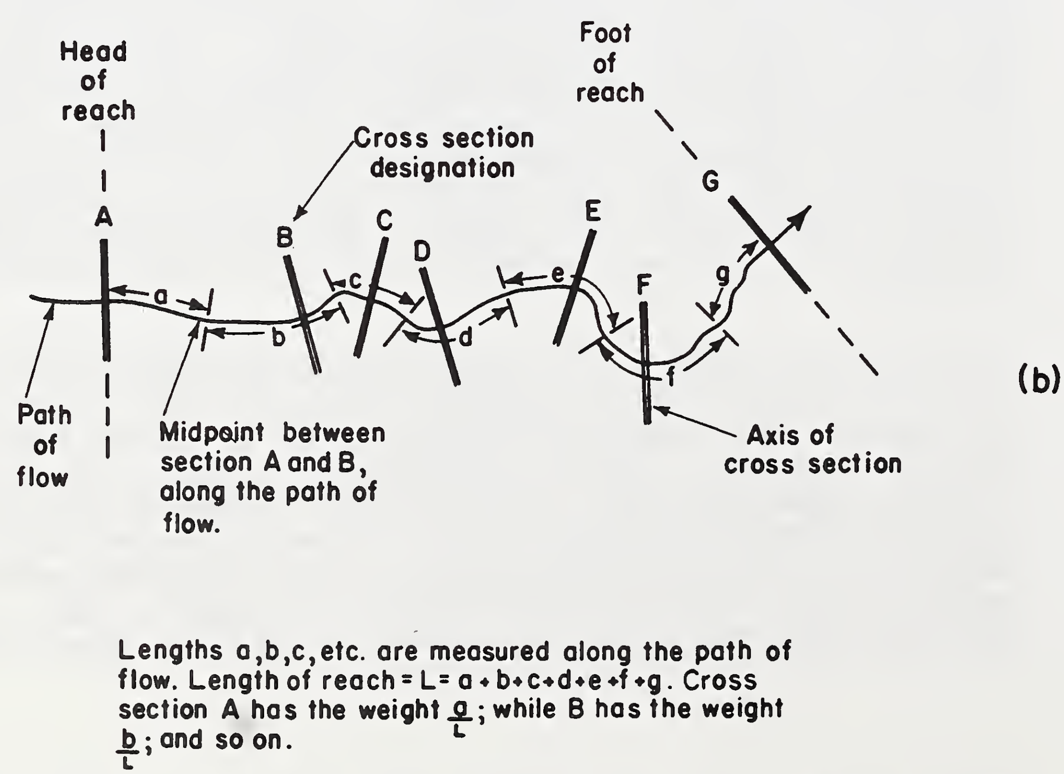

Figure 13-1(b) Flood damage reach showing weighting of area between cross sections.

Table 13-3. Sample computation of stage versus area inundated with two cross sections in the reach (head and foot) and drainage areas not significantly different.

Foot of reach		Head of reach		Areas related to foot of reach 1/			
Cross section 1		Cross section 2					
Stage	Top width	Stage	Top width	Stage	Average top width	Average top width minus channel width	Inundated area in reach 2/
(Feet)	(Feet)	(Feet)	(Feet)	(Feet)	(Feet)	(Feet)	(Feet)
10 3/	41	7 3/	30	10 3/	35.5	0	0
12	168	9	125	12	146.5	111.0	10.7
14	646	11	478	14	562.0	526.5	51.0
16	1070	13	786	16	928.0	892.5	86.5

1/ If related to middle of reach, the stages (col. 5) are 8.5, 10.5, 12.5, and 14.5.

2/ Length of valley in reach is 4230 feet, and

$$\frac{4230}{43560} \text{ (col. 7)} = \text{(col. 8)}$$

3/ Bankfull stage.

Table 13-4. Sample computation of stage versus area inundated with two cross sections in the reach (head and foot), and drainage areas at the sections vary significantly.

Cross section A Foot of reach (D.A.=36.0 sq.mi.)			Cross section B Head of reach (D.A.=24.0 sq.mi.)			Areas related to stages at foot of reach (Cross section A)	
Stage (Feet)	Discharge (cfs)	Top width (Feet)	Discharge (cfs)	Top width (Feet)	Average top width (Feet)	Average top width minus channel width (Feet)	Inundated area in reach (Acres)
10	720 1/	41	680 1/	32	36.5	0	0
12	1510	168	1430 2/	141	154.5	118.0	11.1
14	3060	646	2890 2/	362	504.0	467.5	43.7
16	5030	1070	4750 2/	858	964.0	927.5	87.0

1/ Bankfull discharge.

2/ Proportioned by the bankfull discharge ratio 680/720. For example,

$$\frac{680}{720}(1510) = 1430 \text{ cfs}$$

Length of reach 4080 feet.

frequency. For example, the top width for the 2-year frequency discharge at the upper section is averaged with the top width for the 2-year frequency discharge at the lower section, and so on. When this frequency method is not used and the channel sections vary widely, much accuracy in the averaging should not be expected.

With more than two cross sections, a system of weighting must be used. Figure 13-1b shows a typical reach with seven cross sections on it. The weight for section A is a/L, for section B it is b/L, and so on. Table 13-5 shows a computation using three cross sections. The method of table 13-2 is used to complete the work.

Planimetering method

This procedure can be used either to develop a stage vs. area-inundated relation or to check such a relation developed by other methods.

1. Locate the limits of a selected large recent flood at each cross section on aerial photographs (4-inch to the mile preferred).

2. Using a stereoscope, outline the flood plain for this flood.

3. Lay out and match the photographs, and make a tracing of the floodplain outline. Show the cross section locations and details of land use.

4. Planimeter the area flooded in each reach.

5. Compute the area flooded by using the water surface width at each cross section, for each reach, and multiplying by:

$$\frac{\text{reach length in feet}}{43560}$$

6. Compare the planimetered area with the computed area.

$$C_f = \frac{\text{planimetered area}}{\text{computed area}}$$

7. Compute the area for various other floods, using widths as in Step 5, and assuming the flood plain outline increases and decreases parallel to the outline of the selected recent large flood. Use the correction factor of Step 6, if required.

Table 13-5. Sample computation of stage versus area inundated with three cross sections in the reach and drainage areas at the sections not significantly different.

Cross section 1		Cross section 2		Cross section 3		Related to cross section 1		
Weight = 0.22		Weight = 0.47		Weight = 0.31		Weighted top width	Weighted top width minus channel width	Inundated area in reach
Stage (Feet)	Top width (Feet)	Stage (Feet)	Top width (Feet)	Stage (Feet)	Top width (Feet)	(Feet)	(Feet)	(Acres)
8 1/	42	10 1/	44	7 1/	32	39.8 2/	0	0
10	154	12	250	9	140	194.8	155.0	30.7
12	702	14	540	11	603	595.2	555.4	109.9
14	1100	16	832	13	948	926.9	887.1	175.5

1/ Bankfull stage. Widths at this stage are channel widths.

2/ 39.8 = 0.22 (42) + 0.47 (44) + 0.31 (32). The weights are in proportion to total reach length as shown on figure 13-1b.

Length of reach = 8620 ft.

8. Plot area flooded versus stage at the selected cross section.

9. Determine areas flooded at required depth increments (table 13-2).

Other methods involving planimetering are sometimes useful. For example, flood lines for each of several floods may be used to define inundated areas on aerial photos, which are planimetered and related to stage or runoff or frequency. Generally, lack of data on the location of the flood lines of historic floods limits the application of this and similar methods.

Flood Peak or Volume Versus Area Inundated Method

This method is generally used with alluvial fan floods, although it can also be used instead of the stage methods described above.

1. Make field interviews (the economist usually does this) to determine the areas flooded, for as many floods as possible.

2. Determine actual or estimated flood peak or volume for each flood, using a cross section or gage upstream from the fan as a reference point.

3. Plot the flooded area, in acres, versus the flood peak or volume for each flood, using arithmetic paper. Draw the relation between area and peak or volume.

Once the relation is determined, the effects of upstream projects can be computed in terms of runoff. A reduced runoff means a reduced area flooded. When a channel system within the fan is proposed for reducing flooding, hydrographs are prepared at the upstream section or gage and routed downstream.

Frequency Versus Area Inundated Method

This method is sometimes used instead of the methods described above. It is applicable to both stream reaches and alluvial fans.

1. Determine the area flooded for all known floods by field interview. The earliest known flood determines the length of record, y.

2. Array the "area flooded" values in order of size, the largest first.

3. Refer to Chapter 18 to get frequency plotting positions and tabulate these next to the array for convenience in plotting.

4. Arrange arithmetic graph paper with convenient scales for "area flooded" on the vertical axis and plotting positions on the horizontal axis.

5. Plot the "area flooded" values versus their plotting positions. The point for zero area is determined by field studies.

6. Draw the frequency versus area curve. The area under the curve divided by y gives the average area flooded.

A major objection to this method is that the dollar damage per acre may vary greatly from flood to flood. In such cases, it is more accurate to use a damage-frequency curve.

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SECTION 4

HYDROLOGY

CHAPTER 14. STAGE-DISCHARGE RELATIONSHIPS

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CHAPTER 14. STAGE-DISCHARGE RELATIONSHIPS

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CHAPTER 14. STAGE DISCHARGE RELATIONS

Introduction

In planning and evaluating the structural measures of watershed protection, it is necessary for SCS engineers and hydrologists to develop stage discharge curves at selected locations on natural streams.

Many hydraulics textbooks and handbooks, as well as NEH-5, contain methods for developing stage discharge curves assuming non-uniform steady flow. Some of these methods are elaborate and time consuming. The type of available field data and the use to be made of these stage discharge curves should dictate the method used in developing the curve.

This chapter presents alternate methods of developing these curves at selected points on a natural stream.

Manning's formula has been used to develop stage discharge curves for natural streams assuming the water surface to be parallel to the slope of the channel bottom. This can lead to large errors, since this condition can only exist in long reaches having the same bed slope without a change in cross section shape or retardance.

This condition does not exist in natural streams.

The rate of change of discharge for a given portion of the stage discharge curves differs between the rising and falling sides of a hydrograph. Some streams occupy relatively small channels during low flows, but overflow onto wide flood plains during high discharges. On the rising stage the flow away from the stream causes a steeper slope than that for a constant discharge and produces a highly variable discharge with distance along the channel. After passage of the flood crest, the water re-enters the stream and again causes an unsteady flow, together with a stream slope less than that for a constant discharge. The effect on the stage-discharge relation is to produce what is called a loop rating for each flood.[1/] Generally in the work performed by the SCS the maximum stage the water reached is of primary interest. Therefore, the stage discharge curve used for routing purposes is a plot for the maximum elevation obtained during the passage of flood hydrographs of varying magnitudes. This results in the plot being a single line.

1/ Handbook of Applied Hydrology, Ven Te Chow, page 15-37.

Development of Stage Discharge Curves

Direct Measurement

The most direct method of developing stage discharge curves for natural streams is to obtain velocities at selected points through a cross section. The most popular method is to use a current meter though other methods include the use of the dynamometer, the float, the Pitot tube and chemical and electrical methods. From these velocities and associated cross sectional areas, the discharge is computed for various stages on the rising and falling side of a flood flow and a stage discharge curve developed.

The current meter method is described in detail in USGS Water Supply Paper 888, "Stream Gaging Procedure", and in "Handbook of Hydraulics," by King and Brater, McGraw-Hill, 1963, Fifth edition (generally referred to as King's Handbook).

The velocity head rod (Figure 14-1) may be used to measure flows in small streams or baseflow in larger streams. In making a measurement with a velocity head rod, a tape is stretched across the flowing stream, and both depth and velocity head readings are taken at selected points that represent the cross section of the channel. Table 14-1 is an example of a discharge determined by the velocity head rod. The data is tabulated as shown in columns 1, 2 and 3 of the table and the computation made as shown.

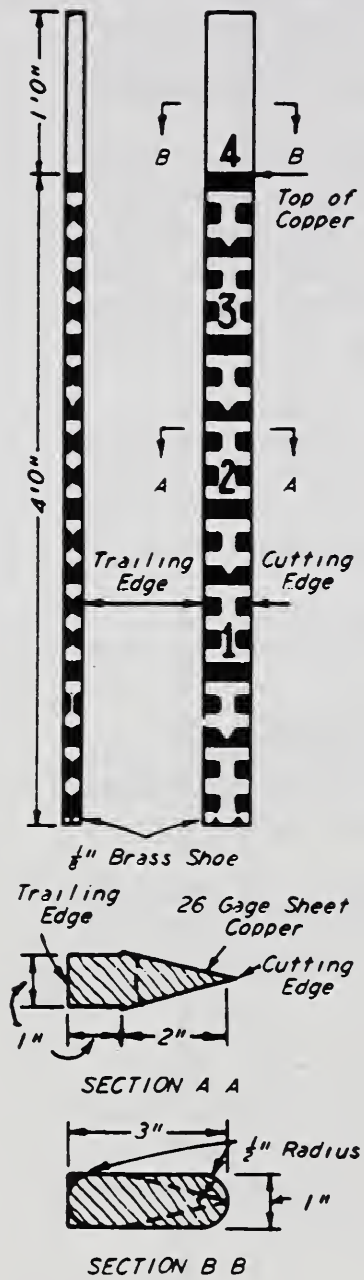
The total area of flow in the section is shown in column 9 and the total discharge in column 10. The average velocity is 45.19/15.00 or 3.01 ft/sec.

Indirect Measurements

Indirectly, discharge is measured by methods such as slope-area, contracted-opening, flow over dam, flow through culvert, and critical depth. These methods, which are described in "Techniques of Water Resources Investigations of the United States Geological Survey," Book 3, Chaps. 3-7, utilize information on the water-surface profile for a specific flood peak and the hydraulic characteristics of the channel to determine the peak discharge.

It should be remembered that no indirect method of discharge determination can be of an accuracy equal to a meter measurement.

Fairly accurate discharges may be computed from measurements made of flows over different types of weirs by using the appropriate formula and coefficients selected from King's "Handbook of Hydraulics," Sections 4 and 5. Overfall dams or broad-crested weirs provide an excellent location to determine discharges. Details on procedures for broad-crested weirs may be found in King's Handbook or USGS Water Supply Paper No. 200, entitled "Weir Experiments, Coefficients, and Formulas" by R. E. Horton.



VELOCITY HEAD ROD
Developed at San Dimas Experimental Forest

The rod is first placed in the water with its foot on the bottom and the sharp edge facing directly upstream. The stream depth at this point is indicated by the water elevation at the sharp edge, neglecting the slight ripple or bow wave. If the rod is now revolved 180 degrees, so that the flat edge is turned upstream a hydraulic jump will be formed by the obstruction to the flow of the stream. After the depth or first reading has been subtracted from the second reading, the net height of the jump equals the actual velocity head at that point. Velocity can then be computed by the standard formula,

$$v = \sqrt{2gh} = 8.02\sqrt{h}$$

in which v = Velocity in ft. per sec.
g = Acceleration of gravity (32.16 ft. per sec. per sec.)
h = Velocity head, in ft.

The average discharge for the stream is obtained by taking a number of measurements of depth and velocity throughout its cross section. $Q = AV$. in which Q = discharge cfs; A = cross sectional area, sq. ft. V = velocity, ft. per sec.

VELOCITY FOR DIFFERENT VALUES OF "h"

$$v = 8.02\sqrt{h}$$

h, Velocity Head in Ft.	Velocity, Ft. per sec.
.05	1.8
.10	2.5
.15	3.1
.20	3.6
.25	4.0
.30	4.4
.35	4.8
.40	5.1
.45	5.4
.50	5.7

Figure 14-1. Velocity head rod for measuring stream flow.

Table 14-1. Computation of discharge using Velocity Head Rod (VHR) measurements.

Distance along Section (ft.)	Depths of flow using VHR: Cut-ting edge (ft.)	Depths of flow using VHR: Flat edge (ft.)	Δh Col 3 - Col 2	Velocity: At point[1] (fps)	Velocity: Average for section (fps)	Mean depth (from Col 2) (ft.)	Width (from Col 1) (ft.)	Area (Col 7 x Col 8) (ft.^{2})	Discharge (Col 9 x Col 6) (cfs)
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)
3.5	0	0		0					
					0.9	0.68	1.00	0.68	0.61
4.5	1.35	1.40	.05	1.8					
					2.15	2.05	0.40	0.82	1.76
4.9	2.75	2.85	.10	2.5					
					3.35	2.90	1.00	2.90	9.72
5.9	3.05	3.32	.27	4.2					
					4.05	3.03	1.50	4.54	18.39
7.4	3.01	3.25	.24	3.9					
					3.40	2.60	.50	1.30	4.42
7.9	2.18	2.31	.13	2.9					
					2.35	1.48	2.80	4.14	9.75
10.7	.78	.83	.05	1.8					
					.90	.39	1.60	.62	.56
12.3	0	0		0					
							Totals	15.00	45.19

[1]/ Column 5 is read from Figure 14-1 using the Δh in column 4.

Slope-Area Estimates

Field measurements taken after a flood are used to determine one or more points on the stage-discharge curve at a selected location. The peak discharge of the flood is estimated using high water marks to determine the slope.

Three or four cross sections are usually surveyed so that two or more independent estimates of discharge, based on pairs of cross sections, can be made and averaged. Additional field work required for slope-area estimates consists of selecting the stream reach, estimating "n" values and surveying the channel profile and high water profile at selected cross sections. The work is guided by the following:

1. The selected reach is as uniform in channel alignment, slope, size and shape of cross section, and factors affecting the roughness coefficient "n" as is practicable to obtain. The selected reach should not contain sudden breaks in channel bottom grade, such as shallow drops or rock ledges.

2. Elevations of selected high water marks are determined on both ends of each cross section.

3. The three or more cross sections are located to represent as closely as possible the hydraulic characteristics of the reach. Distances between sections must be long enough to keep small the errors in estimating stage or elevation.

The flow in a channel reach is computed by one of the open-channel formulas. The most commonly used formula in the slope area method is the Manning equation

$$Q = \frac{1.49}{n} AR^{2/3}S^{1/2} \qquad \text{(Eq. 14-1)}$$

Where Q is the discharge, n is the coefficient of roughness, A is the cross sectional area, R is the hydraulic radius, and S is the slope of the energy gradient. Rearranging Eq. 14-1 gives

$$\frac{Q}{S^{1/2}} = \frac{1.49}{n} AR^{2/3} \qquad \text{(Eq. 14-2)}$$

The right side of Eq. 14-2 contains only the physical characteristics of the cross section and is referred to as the conveyance factor Kd. The slope is determined from the elevations of the highwater mark and the distances between the high water marks along the direction of flow.

Modified Slope Area Method

The following equations based on Bernoulli's theorem are discussed fully in NEH-5, Supplement A.

$$\frac{q^2}{2g} = \frac{E_1 - E_2}{U_2^{+} - U_1^{-}} \qquad \text{(Eq. 14-3)}$$

where

q = discharge, in cfs
E_1 = elevation of the water surface at the upstream section
E_2 = elevation of the water surface at the downstream section
U_2^+ = and U_1^- = symbols used by Doubt for certain computed values; (See NEH-5, page A.14)

The working equation is derived from equation 14-1,

$$q = \left(\frac{2g(E_1 - E_2)}{U_2^+ - U_1^-} \right)^{1/2} \qquad \text{(Eq. 14-4)}$$

Also from NEH-5 -

$$U_2^+ = \frac{1}{a_2^2} + \frac{\ell g\, s_o}{q_{n,d_2}^2} \qquad \text{(Eq. 14-5)}$$

and:

$$U_1^- = \frac{1}{a_1^2} - \frac{\ell g\, s_o}{q_{n,d_1}^2} \qquad \text{(Eq. 14-6)}$$

where ℓ is the length of the reach between sections 1 and 2, and the other symbols are as defined in NEH-5. The nomographs shown in NEH-5, Supplement A as standard drawings ES-75, 76, and 77 are expedient working tools used to solve Equations 14-4, 14-5 and 14-6.

The following example illustrates the modified slope area method and the use of Eq. 14-2. The example is based on data taken from USGS Water Supply Paper 816 (Major Texas Floods of 1936).

Example 14-1 - Using data for the Concho River near San Angelo, Texas, for the September 17, 1936, flood compute the peak discharge that occurred. Figure 14-2 shows Section A and B with the high water mark profile along the stream reach between the two sections.

1. Draw a water surface through the average of the high water mark. From Figure 14-2 the elevation of the water surface at the lower cross section B is 55.98 designated in the example as E_2. The elevation of the water surface at cross section A is 56.50 designated as E_1.

2. Compute the length of reach between the two sections. From Figure 14-2 the length of reach is 680 feet.

3. Divide each cross section into segments as needed due to different "n" values as shown in Figure 14-2.

   In computing the hydraulic parameters of a cross section on a natural stream when flood plain flow exists, it is desirable to divide the cross section into segments. The number of segments will depend on the irregularity of the cross section and

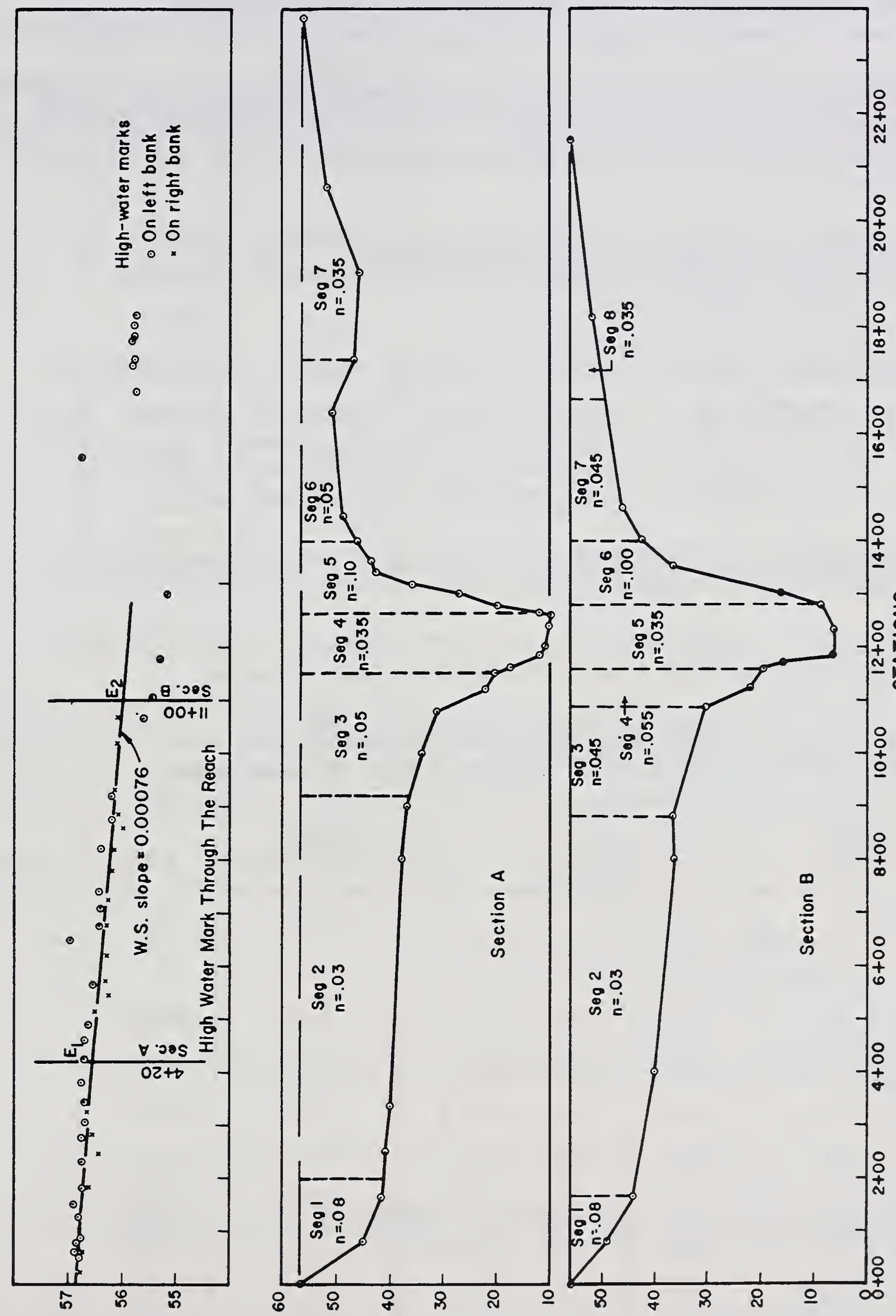


Figure 14-2. High water mark profile and cross sections, Concho River near San Angelo, Texas. Example 14-1.

the variation in "n" values assigned to the different portions. NEH-5, supplement B, gives a method of determining "n" values for use in computing stage discharge curves.

4. Compute the cross sectional area and wetted perimeter for each segment of each cross section. Tabulate in columns 2 and 3 of Table 14-2(a) for cross section A and Table 14-2(b) for cross section B.

5. Compute $F = 1.486\ AR^{2/3}$ for each segment. Using standard drawing ES-76 (NEH-5), compute F and tabulate in column 4, Table 14-2(a) and 14-2(b).

6. Compute $Q/S^{1/2} = 1.486\ AR^{2/3}$. Tabulate the "n" value assigned to each segment in column 5 of Table 14-2(a) and 14-2(b). Column 6 is $A/S^{1/2}$ and is computed by dividing column 4 by column 5 or by using ES-77 (NEH-5). This is commonly called the flow factor of conveyance and is generally designated as Kd.

7. Compute the total area and the total Kd. Sum columns 2 and 6 of Table 14-2(a) and 14-2(b).

8. Compute U^-. Using Eq. 14-6 or ES-77 compute U^- for the downstream cross section A using data from Table 14-2(a).

$$\text{From Eq. 14-6: } U^- = \frac{1}{a_1^2} - \frac{lgs}{q_1^2}$$

$$\frac{1}{a_1^2} = \frac{1}{(34729)^2} = 8.29 \times 10^{-10}$$

$$\frac{s}{q_1^2} = \frac{1}{(91.88 \times 10^5)^2} = 1.18 \times 10^{-14}$$

$$lg\left(\frac{s}{q_1^2}\right) = (680)\ (32.2)\ (1.18 \times 10^{-14}) = 2.58 \times 10^{-10}$$

$$U^- = (8.29 \times 10^{-10}) - (2.58 \times 10^{-10}) = 5.71 \times 10^{-10}$$

9. Compute U^+ Using Eq. 14-5 or ES-77 compute U^+ for upstream cross section B using data in Table 14-2(b).

$$\frac{1}{a_2^2} = \frac{1}{(32771)^2} = 9.31 \times 10^{-10}$$

$$\frac{s}{q_2^2} = \frac{1}{(87.11 \times 10^5)^2} = 1.32 \times 10^{-14}$$

Table 14-2(a) Data for computing discharge from modified slope-area measurements; Cross Section A at Station 4+20. Example 14-1

Segment	Area	Wetted Perimeter	F	n	$\frac{q}{s_o^{1/2}}$
(1)	(2)	(3)	(4)	(5)	(6)
1	2354	252	1.55×10^4	0.080	1.94×10^5
2	12691	735	12.60×10^4	.030	42.00×10^5
3	5862	231	7.50×10^4	.050	15.00×10^5
4	5385	167	8.1×10^4	.035	23.14×10^5
5	2523	135	2.64×10^4	.100	2.64×10^5
6	2498	350	1.38×10^4	.050	2.76×10^5
7	3416	645	1.54×10^4	.035	4.40×10^5
	34729				91.88×10^5

Table 14-2(b) Data for computing discharge from modified slope-area measurements; Cross Section B at Station 11+100. Example 14-1

Segment	Area	Wetted Perimeter	F	n	$\frac{q}{s_o^{1/2}}$
(1)	(2)	(3)	(4)	(5)	(6)
1	1598	236	0.85×10^4	0.080	1.06×10^5
2	11750	725	11.18×10^4	.030	37.27×10^5
3	4750	227	5.37×10^4	.045	11.93×10^5
4	2486	78	3.71×10^4	.055	6.75×10^5
5	4944	153	7.43×10^4	.035	21.23×10^5
6	3455	134	4.47×10^4	.100	4.47×10^5
7	2270	273	1.38×10^4	.045	3.07×10^5
8	1518	513	0.465×10^4	.035	1.33×10^5
	32771				87.11×10^5

$$\lg\left(\frac{s}{q_2^2}\right) = (680)\ (32.2)\ (1.32 \times 10^{-14}) = 2.89 \times 10^{-10}$$

$$U^+ = (9.31 \times 10^{-10} + 2.89 \times 10^{-10}) = 12.20 \times 10^{-10}$$

10. <u>Compute q.</u> Using Eq. 14-4. $q = \left(\frac{2g\ (E_1 - E_2)}{U_2^+ - U_1^-}\right)^{1/2}$

$$q = \sqrt{\frac{(2)\ (32.2)\ (56.50 - 55.98)}{(12.20 - 5.71) \times 10^{-10}}} = 10^5 \times \sqrt{\frac{33.3}{6.49}}$$

$q = 2.265 \times 10^5$ or $q = 226{,}500$. This compares with the discharge of 230,000 cfs computed by USGS in Water Supply Paper 816.

<u>Synthetic methods</u>
There are various methods which depend entirely on data which may be gathered at any time. These methods establish a water surface slope based entirely on the physical elements present such as channel size and shape, flood plain size and shape and the roughness coefficient. The method generally used by the SCS is the modified step method.

This method bases the rate of friction loss in the reach on the elements of the upstream cross section. Manning's equation is applied to these elements and the difference in elevation of the water surface plus the difference in velocity head between the two cross sections is assumed to be equal to the total energy loss in the reach. This method, ignoring the changes in velocity head, is illustrated in Example 14-6.

Selecting Reach Lengths

The flow distance between one section and the next has an important bearing on the friction losses between sections. For flows which are entirely within the channel the channel distance should be used. On a meandering stream the overbank portion of the flow may have a flow distance less than the channel distance. This distance approaches but does not equal the floodplain distance due to the effect of the channel on the flow.

From a practical standpoint the water surface is considered level across a cross section. Thus the elevation difference between two cross sections is considered equal for both the channel flow portion and the overbank portion.

It has been common practice to compute the conveyance for the total section then compute the discharge by using a given slope with this conveyance, where the slope used is an average slope between the slope of the channel portion and the overbank portion. The average slope is computed by the formula:

$$S_a = \frac{H}{L_a} \qquad \text{(Eq. 14-7)}$$

where: S_a = average slope of energy gradient in reach
H = elevation difference of the energy level between sections
L_a = average reach length

The reach length L_a can be computed as follows:

$$q_c = Kd_c \times S_c^{1/2} \qquad \text{(Eq. 14-8)}$$

$$q_f = Kd_f \times S_f^{1/2} \qquad \text{(Eq. 14-9)}$$

$$q_t = Kd_t \times S_a^{1/2} \qquad \text{(Eq. 14-10)}$$

where q_c = discharge in channel portion
Kd_c = conveyance in channel portion
S_c = energy gradient in channel portion
q_f = discharge in floodplain portion
Kd_f = conveyance in floodplain portion
S_f = energy gradient in floodplain portion
q_t = total discharge
Kd_t = total conveyance
S_a = average slope of energy gradient

The total discharge in a reach is equal to the flow in channel plus the flow in the overbank.

Then $$q_t = q_c + q_f \qquad \text{(Eq. 14-11)}$$

Substituting from Equations 14-8, 14-9 and 14-10

$$Kd_t \times S_a^{1/2} = Kd_c \times S_c^{1/2} + Kd_f \times S_f^{1/2} \qquad \text{(Eq. 14-12)}$$

Let $S = \frac{H}{L}$

where H = elev. of reach head - elev. of reach foot
L = length of reach

Then substituting into Eq. 14-12 using the proper subscripts

$$Kd_t \times \left(\frac{H}{L_a}\right)^{1/2} = Kd_c \times \left(\frac{H}{L_c}\right)^{1/2} + Kd_f \times \left(\frac{H}{L_f}\right)^{1/2}$$

Divide both sides by $H^{1/2}$

$$\frac{Kd_t}{L_a^{1/2}} = \frac{Kd_c}{L_c^{1/2}} + \frac{Kd_f}{L_f^{1/2}}$$

$$L_a = \left(\frac{Kd_t}{Kd_c/L_c^{1/2} + Kd_f/L_f^{1/2}}\right)^2 \qquad \text{(Eq. 14-13)}$$

If the average reach length is plotted vs. elevation for a section then it is possible to read the reach length directly to use with the Kd for any desired elevation. The data will plot in a form as shown in Figure 14-3.

This procedure is somewhat difficult to use as each time a new elevation is selected for use a new reach length must also be used.

The procedure can be modified slightly and a constant reach length used in all computations.

Multiply both sides of Equation 14-9 by $\left(\frac{S_c}{S_c}\right)^{1/2}$

This gives:

$$q_f \left(\frac{S_c}{S_c}\right)^{1/2} = (Kd_f)(S_c{}^{1/2}) \left(\frac{S_f}{S_c}\right)^{1/2} \qquad \text{(Eq. 14-14)}$$

The $\left(\frac{S_c}{S_c}\right)^{1/2}$ on the left hand side drops out with a value of 1 giving

$$q_f = (Kd_f)(S_c)^{1/2} \left(\frac{S_f}{S_c}\right)^{1/2} \qquad \text{(Eq. 14-15)}$$

S_f and S_c can be represented as follows

$$S_f = \frac{H}{L_f} \quad \text{or} \quad (S_f)^{1/2} = \left(\frac{H}{L_f}\right)^{1/2} \qquad \text{(Eq. 14-16)}$$

$$S_c = \frac{H}{L_c} \quad \text{or} \quad (S_c)^{1/2} = \left(\frac{H}{L_f}\right)^{1/2} \qquad \text{(Eq. 14-17)}$$

Divide Equation 14-16 by Equation 14-17

$$\left(\frac{S_f}{S_c}\right)^{1/2} = \frac{\left(\frac{H}{L_f}\right)^{1/2}}{\left(\frac{H}{L_c}\right)^{1/2}} = \left(\frac{L_c}{L_f}\right)^{1/2} \qquad \text{(Eq. 14-18)}$$

Equation 14-15 becomes by substitution:

$$q_f = (Kd_f)(S_c)^{1/2} \left(\frac{L_c}{L_f}\right)^{1/2} \qquad \text{(Eq. 14-19)}$$

The term $\left(\frac{L_c}{L_f}\right)$ is commonly referred to as the meander factor.

Then substituting Equation 14-19 and 14-8 into Equation 14-11 we get

$$q_t = (Kd_c)(S_c)^{1/2} + (Kd_f)(S_c)^{1/2} \left(\frac{L_c}{L_f}\right)^{1/2}$$

Rearranging we get

$$q_t = \left(Kd_c + (Kd_f)\left(\frac{L_c}{L_f}\right)^{1/2}\right)(S_c)^{1/2} \qquad \text{(Eq. 14-20)}$$

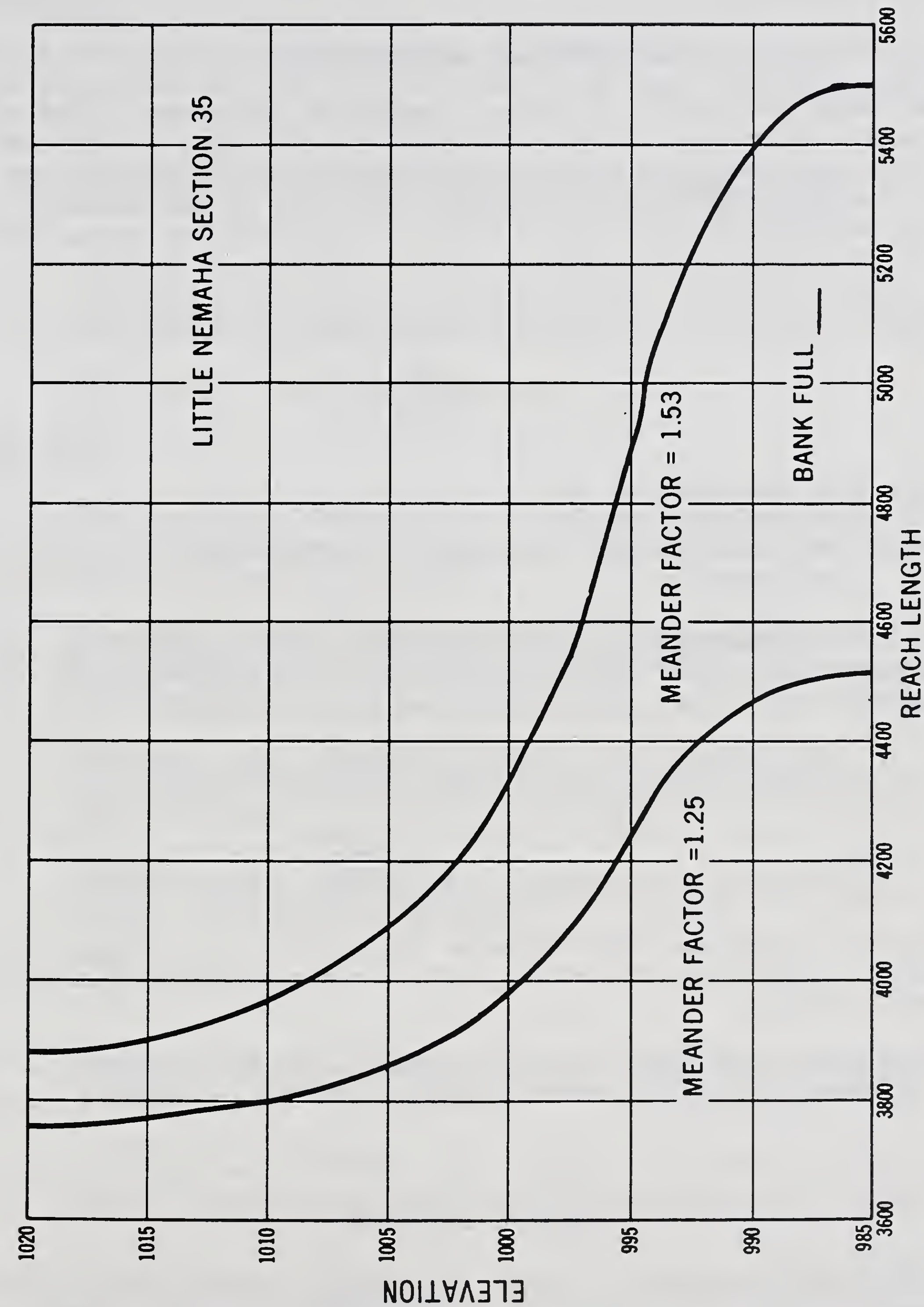


Figure 14-3. Reach length vs. elevation, Little Nemaha Section 35.

Equation 14-20 can be used to compute the total stage discharge at a section by using the channel reach length rather than a variable reach length. Example 14-5 illustrates the use of modifying the flood plain conveyance by the square root of the meander factor in developing a stage discharge curve.

Discharge vs. Drainage Area

It is desirable for the water surface profile to represent a flow which has the same occurrence interval throughout the watershed. The CSM (cubic feet per second per square mile) values for most floods vary within a channel system having a smaller value for larger drainage areas. Thus when running a profile the 50 CSM of the outlet, the actual CSM rate will increase as the profile progresses up the watershed.

The rate of discharge at any point in the watershed is based on the formula[1/]

$$Q = 46C\,A^{\left(\frac{.894}{A^{.048}} - 1\right)} \qquad \text{(Eq. 14-21)}$$

where Q is discharge in CSM
A is the drainage area
and C is a coefficient depending on the characteristics of the watershed

Assuming that C remains constant for any point in the watershed, then the discharge at any point in the watershed may be related to the discharge of any other point in the watershed by the formula

$$\frac{Q_1}{Q_2} = K = \frac{A_1^{\left(\frac{.894}{A_1^{.048}} - 1\right)}}{A_2^{\left(\frac{.894}{A_2^{.048}} - 1\right)}} \qquad \text{(Eq. 14-22)}$$

where Q_1 and A_1 represent the discharge rate in CSM and drainage area of one point in the watershed and Q_2 and A_2 represent the CSM and drainage area at another.

In practice Q_2 and A_2 usually represent the outlet of the watershed and remain constant and A_1 is varied to obtain Q_1 at other points of interest.

Equation 14-22 is plotted in Exhibit 14-1 for the case where A_2 is 400 square miles. This curve may be used directly to obtain the CSM

1/ Engineering For Dams, Vol. 1 page 125, Creager, Justin & Hines.

discharge of the outlet if the outlet is at 400 square miles as shown in Example 14-2. Example 14-3 shows how to use Exhibit 14-1 if the drainage area at the outlet is not 400 square miles.

<u>Example 14-2</u>

Find the CSM value to be used for a reach with a drainage area of 50 square miles when the CSM at the outlet is 80 CSM. The drainage area at the outlet is 400 square miles.

1. <u>Determine K for a drainage area of 50 square miles</u>. From Exhibit 14-1 with a drainage area of 50 square miles read K = 2.61.

2. <u>Determine CSM rate for 50 square miles</u>. Multiply CSM at the outlet by K computed in step 1.

   (80) (2.61) = 209 CSM @ 50 square miles.

<u>Example 14-3</u>

Find the CSM rate to be used at a reach with a drainage area of 20 square miles if the drainage area at the outlet is 50 square miles. The CSM rate at the outlet is 60 CSM.

1. <u>Determine K for a drainage area of 20 square miles</u>. From Exhibit 14-1 for a drainage area of 20 square miles read K = 3.66.

2. <u>Determine K for a drainage area of 50 square miles</u>. From Exhibit 14-1 for a drainage area of 50 square miles read K = 2.61.

3. <u>Compute a new K value for a drainage area of 20 square miles</u>. Divide step 1 by step 2.

   $$\frac{3.66}{2.61} = 1.40$$

4. <u>Determine CSM rate for the 20 square mile drainage area</u>. Multiply K obtained in step 3 by the CSM at the outlet.

   (1.40) (60) = 84 CSM

<u>Computing Profiles</u>

When using water surface profiles to develop stage discharge curves for flows at more than critical depth, it is necessary to have a stage discharge curve for a starting point at the lower end of a reach. This starting point may be a stage discharge curve developed by current meter measurements or one computed from a control section where the flow passes through critical discharge; or it may be one computed from the elements

of the cross section and an estimate of the slope. The latter case is the most commonly used by SCS since the more accurate stage discharge curves are not generally available on small watersheds. In most cases it is advisable to locate three or four cross sections close together in order to eliminate part of the error in estimating the slope used in developing the stage discharge curve at the lower or first cross section on a watershed.

Example 14-4

Develop the starting stage discharge curve for cross section M-1 (Figure 14-4) shown as the first cross section at the outlet end of the watershed, assuming an energy gradient of .001 ft/ft.

1. Plot the surveyed cross section. From field survey notes, plot the cross section, Figure 14-5(a) noting the points where there is an apparent change in the "n" value.

2. Divide the cross section into segments. An abrupt change in shape or a change in "n" is the main factor to be considered in determining extent and number of segments required for a particular cross section. Compute the "n" value for each segment using NEH-5, Supplement B, or the "n" may be based on other data or publications.

3. Plot the channel segment on an enlarged scale. Figure 14-5(b), for use in computing the area and measuring the wetted perimeter at selected elevations in the channel. The length of the segment at selected elevations is used as the wetted perimeter for the flood plain segments. The division line between each segment is not considered as wetted perimeter.

4. Tabulate elevations to be used in making computations. Starting at an elevation equal to or above any flood of record, tabulate in column 1 of Table 14-3 the elevations that will be required to define the hydraulic elements of each segment.

5. Compute the wetted perimeter at each elevation listed in step 4. Using an engineer's scale and starting at the lowest elevation in column 1, measure the wetted perimeter of each segment at each elevation and tabulate in columns 3, 7, 11, and 15 of Table 14-3. Note that the maximum wetted perimeter for the channel segment is 62 at elevation 94.

6. Compute the cross sectional area for each elevation listed in step 4. Starting at the lowest elevation, compute the accumulated cross sectional area for each segment at each elevation in column 1 and tabulate in columns 2, 6, 10, and 14 of Table 14-3.

7. Compute F factor. $F = 1.486AR^{2/3}$ for each elevation. Using standard drawing ES-76, compute the F factor for each segment

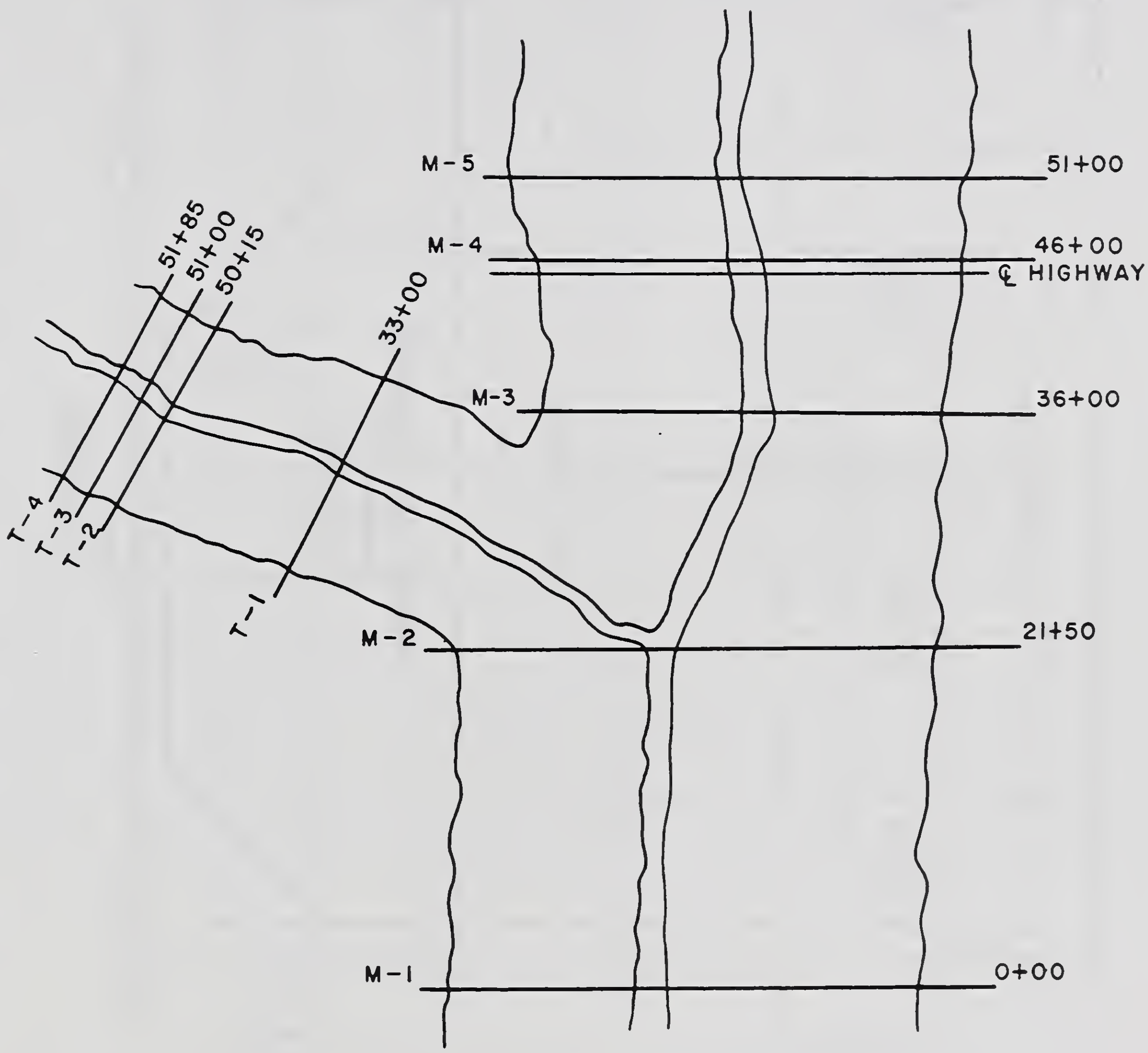


Figure 14-4. Schematic of Watershed for Examples 14-4, 14-5, and 14-6.

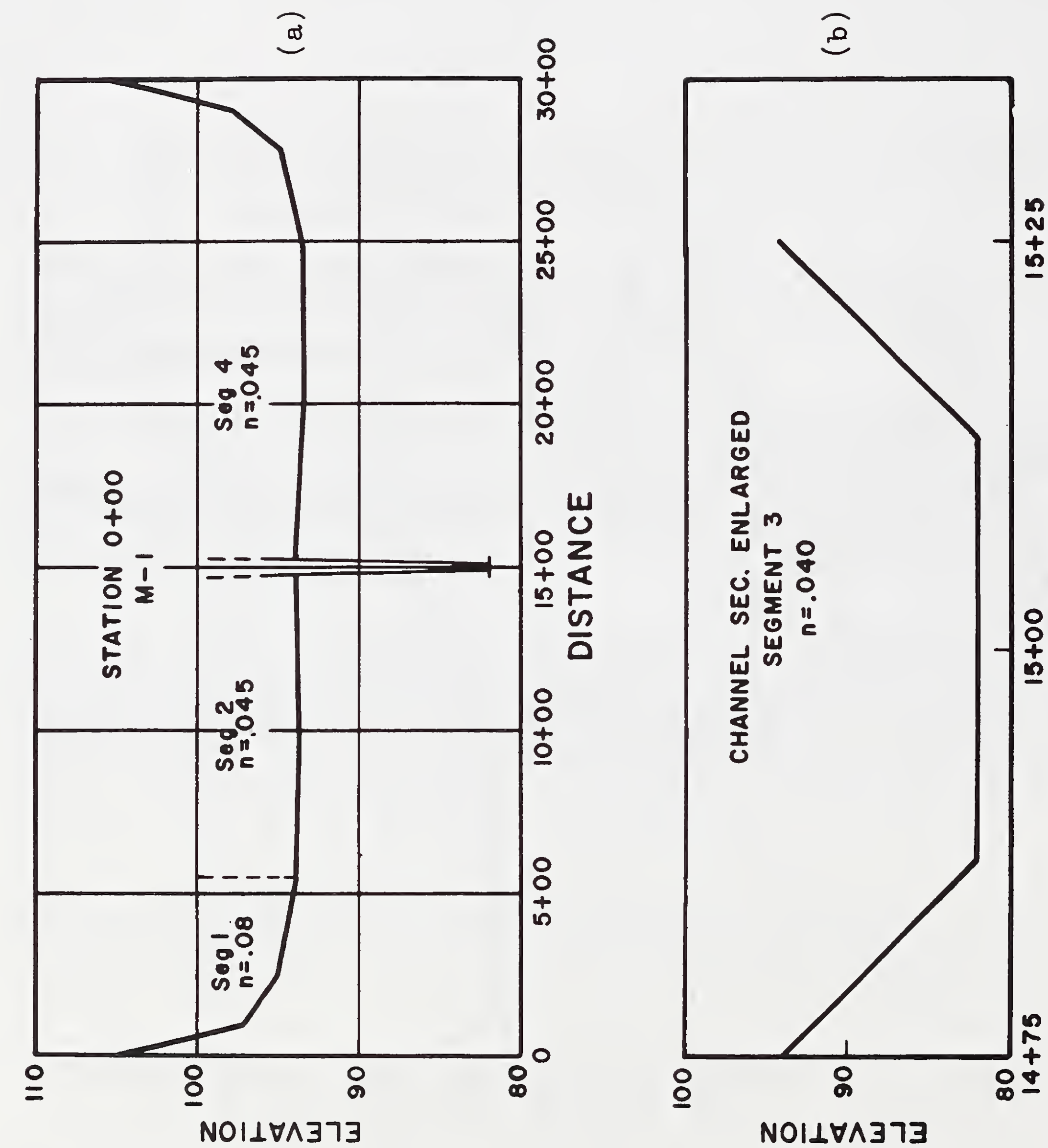


Figure 14-5. Cross section M-1, Examples 14-4 and 14-5.

Table 14-3. Hydraulic parameters for starting cross section M-1, Example 14-4.

	n = .08 Segment 1				n = .045 Segment 2				n = .040 Segment 3				n = .045 Segment 4					
Elev	Area	WP	F	$q_{nd}/S_o^{1/2}$	A	WP	F	$q_{nd}/S_o^{1/2}$	A	WP	F	$q_{nd}/S_o^{1/2}$	A	WP	F	$q_{nd}/S_o^{1/2}$	$\Sigma\ q_{nd}/S_o^{1/2}$	Σ Area
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)	(19)
105	4862	550	30600	3.80×10^5	10268	925	75500	1.67×10^6	1006	62	9560[1/]	2.38×10^5	15555	1460	112000[2/]	2.49×10^6	4.78×10^6	31691
102	3272	510	17000	2.12×10^5	7493	925	44700	9.83×10^5	856	62	7300	1.82×10^5	11175	1440	65000	1.44×10^6	2.81×10^6	22796
100	2272	490	9350	1.17×10^5	5643	925	27900	6.20×10^5	756	62	5940	1.48×10^5	8325	1400	40800	9.06×10^5	1.79×10^6	16996
98	1322	460	3970	4.96×10^4	3793	925	14400	3.20×10^5	656	62	4700	1.17×10^5	5523	1380	20800	4.60×10^5	9.47×10^5	11294
96	487	375	860	1.07×10^4	1943	925	4740	1.05×10^5	556	62	3560	8.9×10^4	2833	1300	7040	1.56×10^5	3.61×10^5	5819
95	150	300	140	1.75×10^3	1018	925	1615	3.59×10^4	506	62	3040	7.6×10^4	1543	1275	2600	5.78×10^4	1.72×10^5	3217
94	0	0	0	0	93	925	30	6.67×10^2	456	62	2560	6.4×10^4	378	1050	284	6.32×10^3	7.10×10^4	927
93					0	0	0	0	407	58	2250	5.61×10^4	0	0	0	0	5.61×10^4	407
91									315	52	1560	3.92×10^4					3.92×10^4	315
89									231	46	1080	2.51×10^4					2.51×10^4	231
87									155	41	560	1.40×10^4					1.40×10^4	155
85									87	35	236	5.90×10^3					5.90×10^3	87
82									0	26	0	0						0

[1/] To solve this on ES-77 divide F by 2, then double results read from Sheet 3, ES-77.
[2/] In order to solve this on ES-76 it is necessary to divide both area and WP by 2 and then double the F factor read from Sheet 3, ES-76.

NOTE: $q_{nd}/S_o^{1/2}$ is the same as Kd or commonly referred to as the conveyance factor.

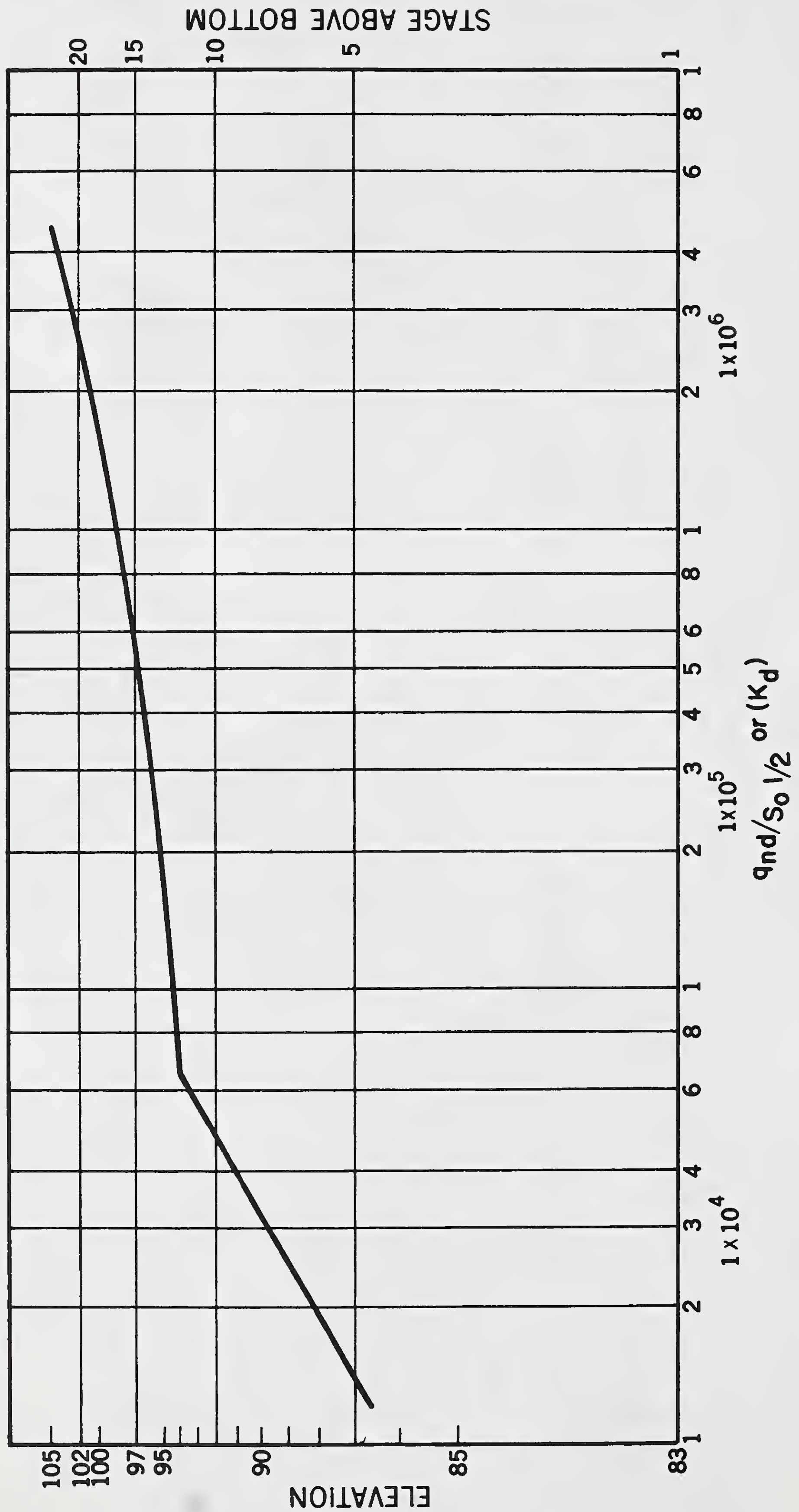


Figure 14-6. Conveyance values section M-1, Example 14-4.

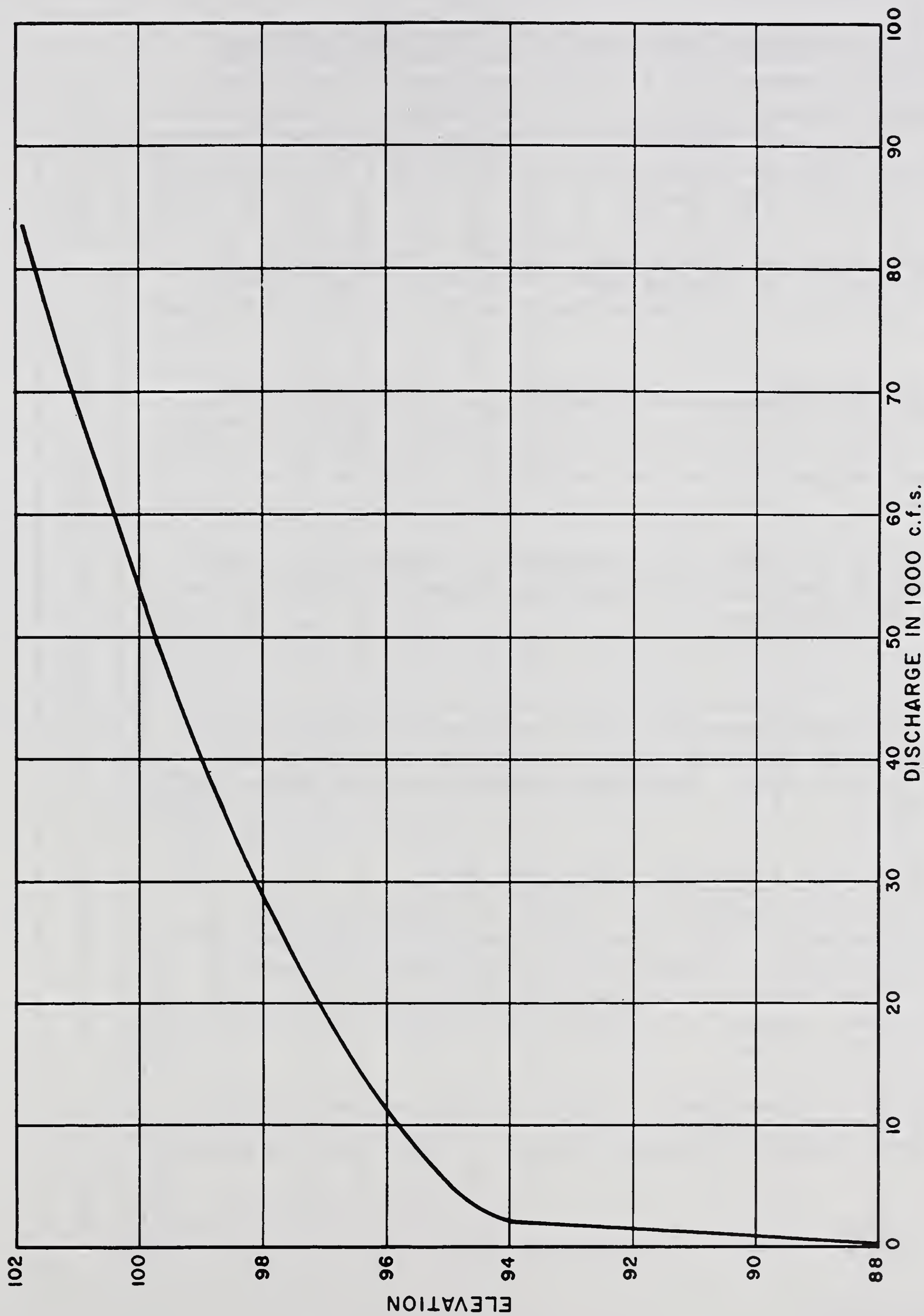


Figure 14-7. Stage discharge section M-1, Example 14-4.

at each elevation in column 1 and tabulate in columns 4, 8, 12, and 16, Table 14-3.

8. Compute the conveyance factor $q_{nd}/S_o^{1/2}$ for each elevation.

Using standard drawing ES-77 and the assigned "n" value for each segment compute $q_{nd}/S_o^{1/2}$ for each segment at each elevation in column 1 and tabulate in columns 5, 9, 13, and 17 of Table 14-3. This can also be done by dividing F by n using a slide rule or desk calculator.

9. Sum columns 5, 9, 13, and 17 and tabulate in column 18. A plot of column 18 on log-log paper is shown on Figure 14-6. The elevation scale is selected based on feet above the channel bottom.

10. Compute the discharge for each elevation. Using the average slope at cross section M-1, S = .001, develop stage discharge for cross section M-1, $q = S^{1/2} \times q_{nd}/S_o^{1/2}$, or $q = S^{1/2} \times Kd$. The stage discharge curve for cross section M-1 is shown on Figure 14-7.

The next example shows the effect of a meandering channel in a floodplain on the elevation discharge relationship. Equation 14-20 will be used to determine the discharge.

Example 14-5

Develop the stage discharge curve for cross section M-1 (Figure 14-4) if M-1 represents a reach having a channel length of 2700 feet and a floodplain length of 2000 feet. The energy gradient of the channel portion is 0.001 ft./ft.

1. Compute the total floodplain conveyance Kd_f.

Figure 14-5 shows segments 1, 2 and 4 of section M-1 are floodplain segments. Table 14-3 of Example 14-4 was used to develop the hydraulic parameters for section M-1 for each segment. From Table 14-3 add the $Qnd/So^{1/2}$ values for each elevation from columns 5, 9, and 17 and tabulate as Kd_f in column 2 of Table 14-4.

2. Determine the meander factor L_c/L_f. For the channel length of 2700 feet and the floodplain length of 2000 feet the meander factor is:

$$\frac{2700}{2000} = 1.35$$

3. Determine $L_c/L_f^{1/2}$.

$$(1.35)^{1/2} = 1.16$$

Table 14-4. Stage discharge for Section M-1 with meander correction, Example 14-5

Elevation	Floodplain Kd_f	$Kd_f \left(\frac{I_c}{L_f}\right)^{1/2}$	Channel Kd_c	Col. 3 + Col. 4	Discharge Q_t
(1)	(2)	(3)	(4)	(5)	(6)
105	4.54×10^6	5.27×10^6	2.38×10^5	5.51×10^6	174000
102	2.64×10^6	3.06×10^6	1.82×10^5	3.24×10^6	102000
100	1.64×10^6	1.90×10^6	1.48×10^5	2.05×10^6	64800
98	8.30×10^5	9.63×10^5	1.17×10^5	1.08×10^6	34100
96	2.72×10^5	3.16×10^5	8.9×10^4	4.05×10^5	12800
95	9.55×10^4	1.11×10^5	7.6×10^4	1.87×10^5	5910
94	6.99×10^3	8.11×10^3	6.4×10^4	7.21×10^4	2280
93	0.	0.	5.61×10^4	5.61×10^4	1770
91	0.	0.	3.92×10^4	3.92×10^4	1240

4. Compute $(Kd_f)\ (L_c/L_f)^{1/2}$. For each elevation in column 1 of Table 14-4 multiply column 2 by $(L_c/L_f)^{1/2}$ and tabulate in column 3.

$$(4.54 \times 10^6)\ (1.16) = 5.27 \times 10^6$$

5. Compute the channel conveyance Kd. From Figure 14-4 the channel is segment 3 and the conveyance has been calculated in column 13 of Table 14-3. Tabulate Kd_c in column 4 of Table 14-4.

6. Compute $Kd_c + (Kd_f)\ (L_c/L_f)^{1/2}$. From Table 14-4 add columns 3 and 4 and tabulate in column 5.

7. Compute the discharge for each elevation. Use $S_c = .001$ and Equation 14-20. Multiply columns by $S_c{}^{1/2}$ and tabulate in column 6.

$$Q_t = (Kd_c + (Kd_f)\ (L_c/L_f)^{1/2})\ (S_c)^{1/2}$$

$$Q_t = (5.51 \times 10^6)\ (3.16 \times 10^{-2}) = 1.74 \times 10^5 = 174{,}000 \text{ cfs.}$$

The next example will show the use of the modified step method in computing water surface profiles. It is a trial and error procedure based on estimating the elevation at the upstream section, determining the conveyance, Kd, for the estimated elevation and computing $S^{1/2}$ by using Mannings equation in the form $S^{1/2} = \frac{Q}{Kd}$ where $Kd = \frac{1.486}{n} AR^{2/3}$. S is the head loss per foot (neglecting velocity head) from the downstream to the upstream section. This head loss added to the downstream water surface elevation should equal the estimated upstream elevation.

Example 14-6

Using the rating curve developed in Example 14-4 for cross section M-1 and parameters plotted on Figures 14-8 and 14-10 for cross sections M-2 and T-1, compute the water surface profiles required to develop stage discharge curves for cross sections M-2 and T-1. The changes in velocity head will be ignored for these computations. The drainage area at section M-1 is 400 sq. mi., at M-2 is 398 sq. mi. and at T-1 is 48 sq. mi. The reach length between M-1 and M-2 is 2150 feet and between M-2 and T-1 is 1150 feet. Assume the meander factor for this example is 1.0.

1. Determine the range of csm needed to define the stage discharge curve. One or more of the csm's selected should be contained within the channel. Tabulate in column 1, Table 14-5(a).

2. Compute the discharge in cfs for each csm at the two cross sections M-1 and M-2. At section M-1 the drainage area is 400 sq. mi. Using Exhibit 14-1 the K factor is 1.0 and the cfs for 2 csm is 2 x 400 x 1.0 = 800 cfs. At section M-2 the drainage

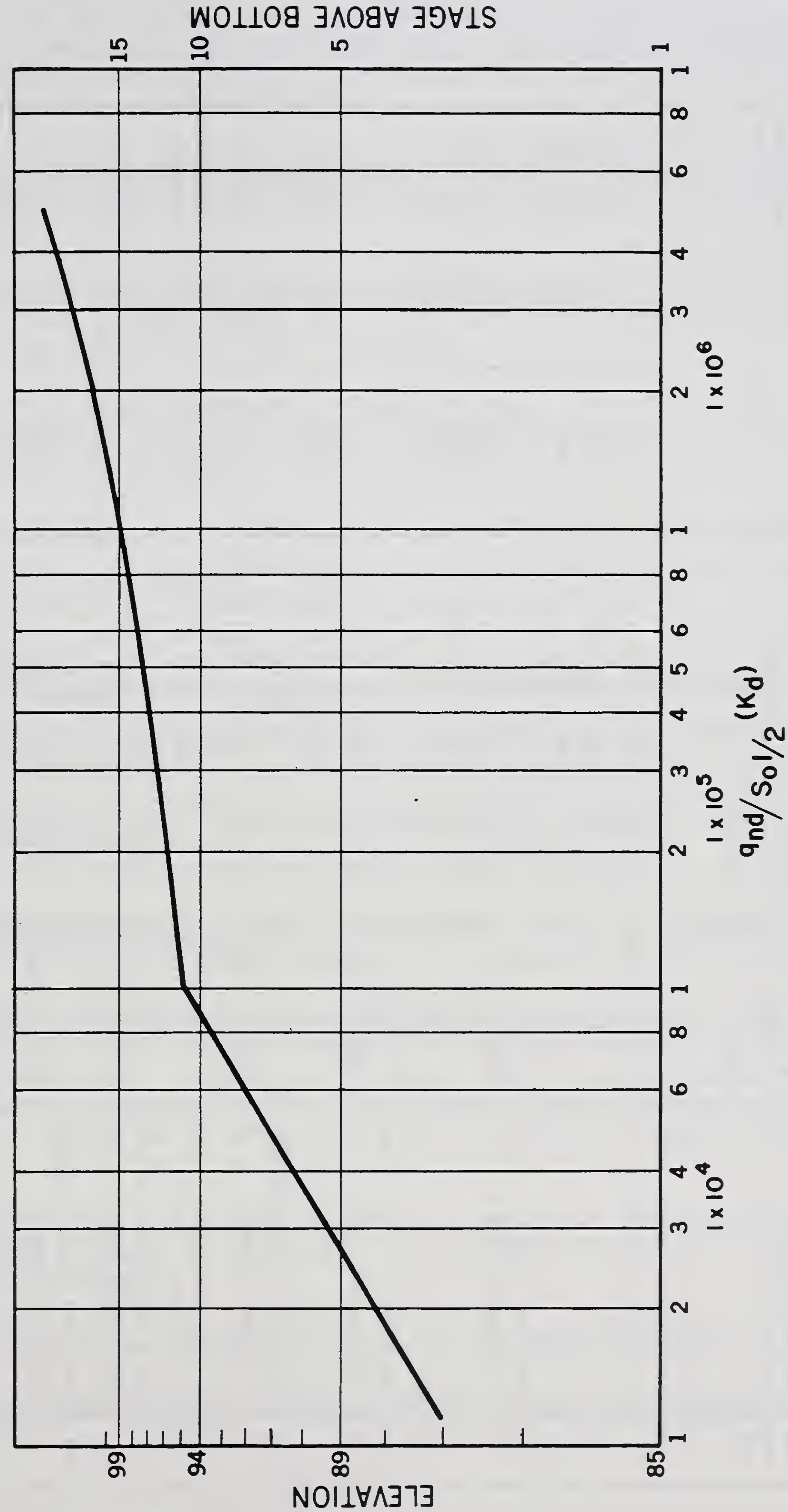


Figure 14-8. Conveyance values section M-2, Example 14-6.

Table 14-5(a). Water Surface profiles from cross section M-1 to M-2, Example 14-6.

CSM	Discharge in cfs M-1	M-2	ℓ_c	Elev. @ M-1	Assumed elev. @ M-2	Kd_{M-2}	$\left(\frac{q_{M-2}}{Kd_{M-2}}\right)^2 = S_f$	$S_f \times \ell$	Col 5 + Col 9 estimate elev @ M-2	Computed elev. @ M-2
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)
2	2 x 400 x 1.000 = 800									
	2 x 398 x 1.002[1] =	798	2150[2]	89.22	90.0	3.70×10^4	.00046	.99	90.21^+	
					90.2	3.89×10^4	.00044	.95	90.17^-	90.2
					90.1	3.75×10^4	.00045	.97	90.19^+	
10	10 x 400 x 1.000 = 4000									
	10 x 398 x 1.002 =	3990	2150	94.66	95.2	1.20×10^5	.00107	2.30	96.96^+	
					95.6	1.60×10^5	.00062	1.34	96.00^+	95.7
					95.8	1.90×10^5	.00044	.95	95.61^-	
20	20 x 400 x 1.000 = 8000									
	20 x 398 x 1.002 =	7980	2150	95.52	96.7	3.50×10^5	.00052	1.12	96.62^-	96.7
					96.6	3.30×10^5	.00059	1.26	96.78^+	
50	50 x 400 x 1.000 = 20000									
	50 x 398 x 1.002 =	19950	2150	97.12	98.3	7.80×10^5	.00065	1.40	98.52^+	
					98.4	8.00×10^5	.00062	1.34	98.46^+	98.4
					98.5	8.20×10^5	.00059	1.27	98.39^-	
100	100 x 400 x 1.000 = 40000									
	100 x 398 x 1.002 =	39900	2150	98.96	100.3	1.60×10^6	.00062	1.33	100.29^-	100.3
200	200 x 400 x 1.000 = 80000									
	200 x 398 x 1.002 =	79800	2150	101.68	103.2	3.20×10^6	.00062	1.33	103.01^-	103.1
					103.1	3.00×10^6	.00071	1.52	103.20^+	

1/ Computed from equation shown on Exhibit 14-1.
2/ Where the channel length is different from the flood plain length, Kd values for flood plain portion of section are modified so channel length may be used in all calculations.

area is 398 sq. mi. and from Exhibit 14-1 the K factor is 1.002. For 2 csm the discharge at M-2 is 2 x 398 x 1.002 = 798 cfs. Tabulate the discharges at M-1 and M-2 on Table 14-5(a), columns 2 and 3 of Table 14-5(a).

3. Tabulate the reach length between the two cross sections in column 4. The reach length between section M-1 and M-2 is 2150 feet.

4. Determine the water surface elevation at M-1. For the discharge listed in column 2 read the elevation from Figure 14-7 and tabulate in column 5 of Table 14-5(a).

5. Assume a water elevation at section M-2. For the smallest discharge of 798 cfs assume an elevation of 90.0 at M-2 and tabulate in column 6 of Table 14-5(a).

6. Determine Kd for assumed elevation. Read $Qnd/So^{1/2}$ or Kd_{M-2} of 3.70×10^4 at elevation 90.0 from Figure 14-8 and tabulate in column 7 of Table 14-5(a).

7. Determine S_f. $S_f = \frac{(Q_{M-2})^2}{(Kd_{M-2})}$. Divide column 3 by column 7 and square the results $(798/37000)^2$ = .00046 and tabulate in column 8 of Table 14-5(a).

8. Determine $S_f \times \ell$. Multiply column 8 by column 4, .00046 x 2150 = .99, and tabulate in column 9 of Table 14-5(a).

9. Compute elevation at M-2. Add column 9 (S_f) to column 5 (elevation at M-1) and tabulate in column 10 of Table 14-5(a).

10. Compare computed elevation with assumed elevation. Compare column 10 with column 6 and adjust column 6 up if column 10 is greater and down if it is less. For 2 csm discharge the computed elevation is 90.12 and the estimated elevation is 90.0. Since column 10 is greater a revision in the estimated elevation at M-2 in column 6 must be made.

    Repeat steps 5 through 10 until a reasonable balance between column 10 and 6 is obtained. A tolerance of 0.1 foot was used in this example.

11. Repeat steps 5 through 10 for each csm value selected.

12. Plot stage discharge curve, columns 3 and 11 as shown on Figure 14-9.

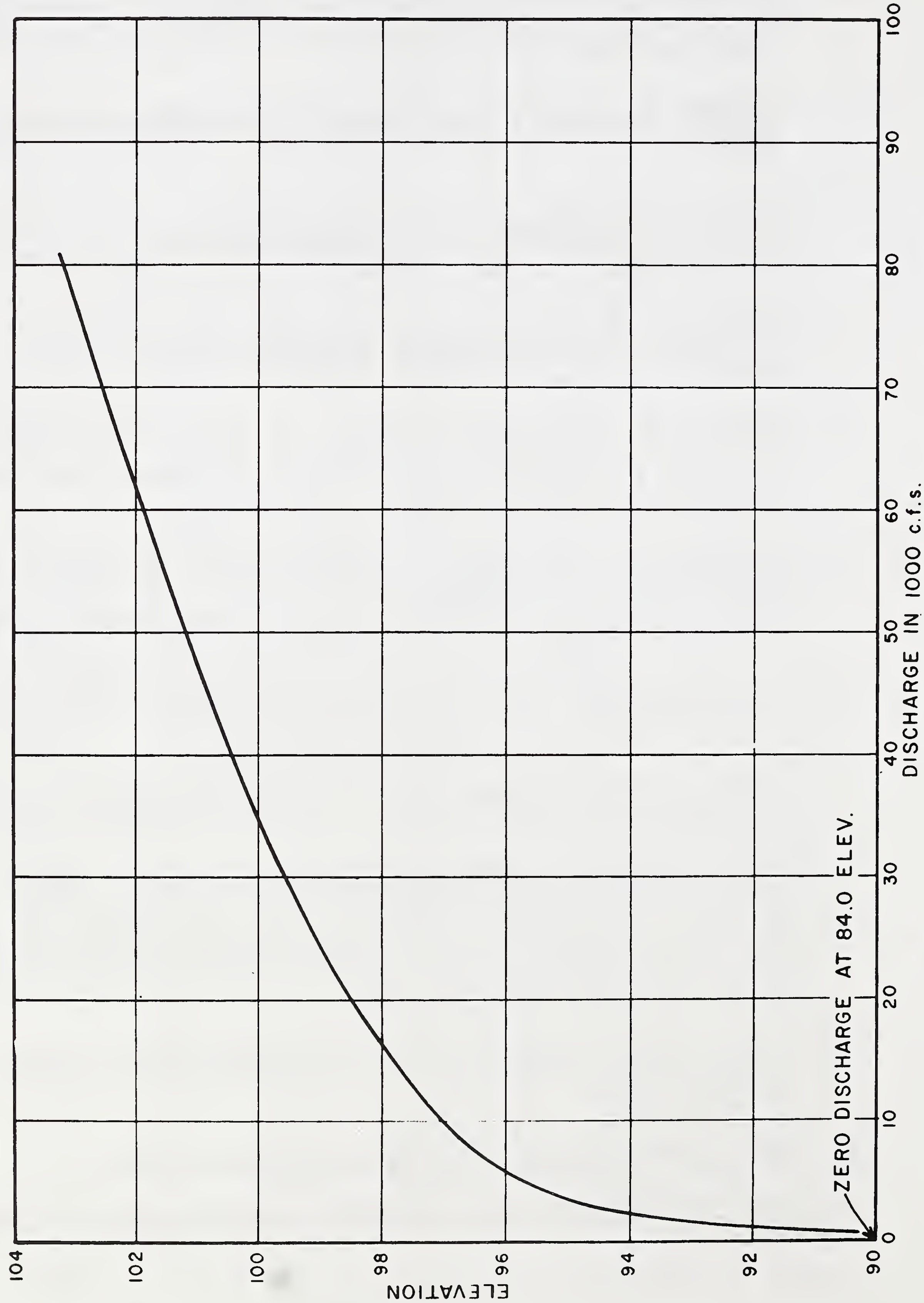


Figure 14-9. Stage discharge section M-2, Example 14-6.

Table 14.5(b). Water surface profiles from cross section M-2 to T-1. Example 14-6.

CSM	Discharge in cfs	M-2	T-1	ℓ_C	Elev. @ M-2	Assumed elev. @ T-1	Kd_{T-1}	$\left(\frac{q_{T-1}}{Kd_{T-1}}\right)^2 = S_f$	$S_f \times \ell$	Col 5 + Col 9 estimate elev @ T-1	Computed elev. @ T-1
(1)		(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)
2	2 x 398 x 1.002 =	798									
	2 x 48 x 2.68 1/ =		260	1150	90.15	93.0	1.7×10^3	.0234	26.9	116.9^+	
						94.0	3.4×10^3	.0058	6.70	96.8^+	
						94.5	4.4×10^3	.0035	4.02	94.2^-	94.4
						94.4	4.2×10^3	.0038	4.42	94.6^+	
10	10 x 398 x 1.002 =	3990									
	10 x 48 x 2.68 =		1290	1150	95.70	97.0	1.65×10^4	.0062	7.05	102.75^+	
						98.0	5.7×10^4	.00051	.59	96.29^-	97.5
						97.5	3.2×10^4	.00163	1.87	97.57^+	
20	20 x 398 x 1.002 =	7980									
	20 x 48 x 2.68 =		2580	1150	96.65	98.0	5.7×10^4	.00205	2.36	99.01^+	
						98.5	9.0×10^4	.00082	.94	97.59^-	98.2
						98.2	7.0×10^4	.00135	1.56	98.21^+	
50	50 x 398 x 1.002 =	19950									
	50 x 48 x 2.68 =		6450	1150	98.45	100.0	2.32×10^5	.00077	.89	99.34^-	
						99.5	1.85×10^5	.00122	1.40	99.85^+	99.7
						99.65	2.00×10^5	.00104	1.20	99.65^-	
100	100 x 398 x 1.002 =	39900									
	100 x 48 x 2.68 =		12900	1150	100.3	101.0	3.6×10^5	.00128	1.47	101.77^+	
						101.2	4.0×10^5	.00104	1.2	101.5^+	101.3
						101.3	4.4×10^5	.00086	.99	101.29^-	
200	200 x 398 x 1.002 =	79800									
	200 x 48 x 2.68 =		25800	1150	103.15	104.0	9.2×10^5	.00078	.90	104.05	104.0

1/ Taken from Exhibit 14-1.

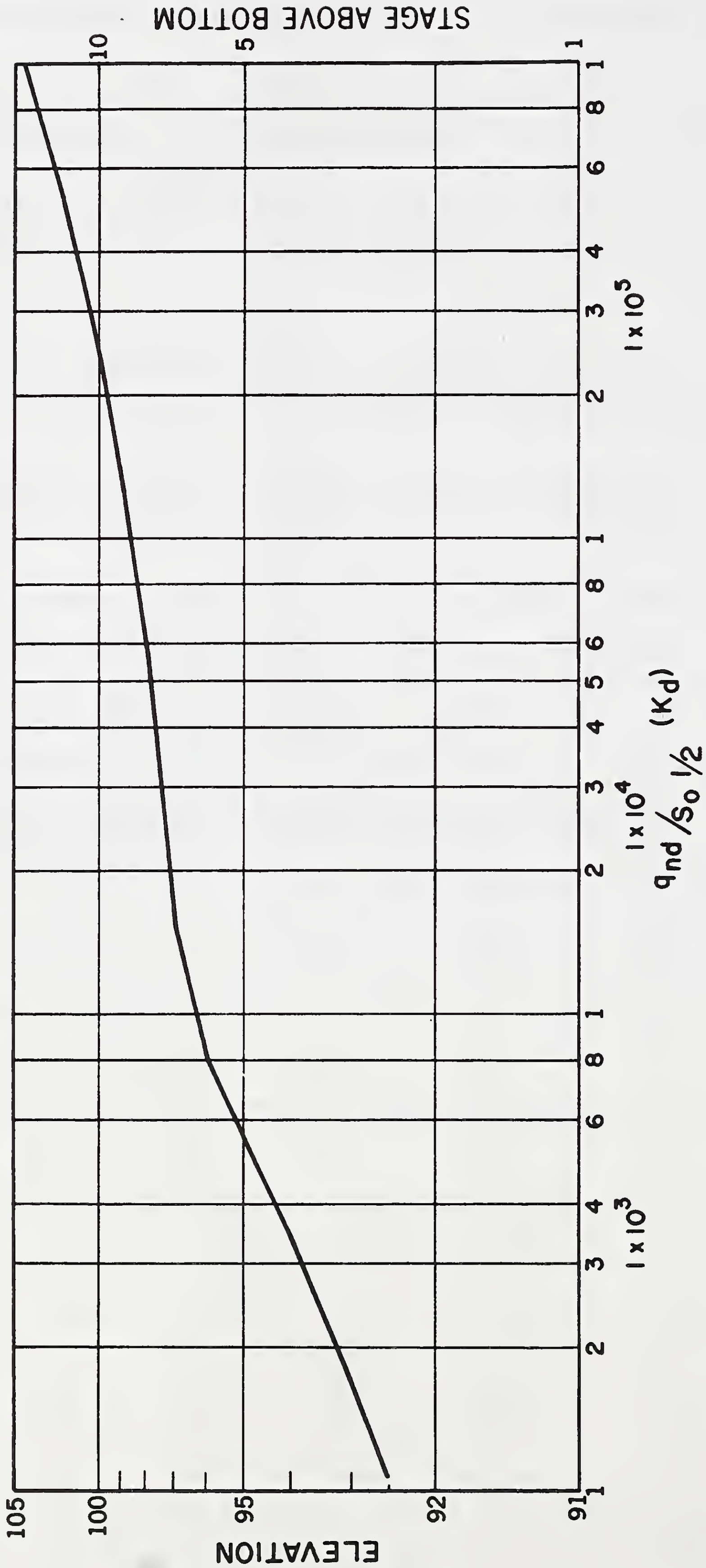


Figure 14-10. Conveyance values section T-1, Example 14-6.

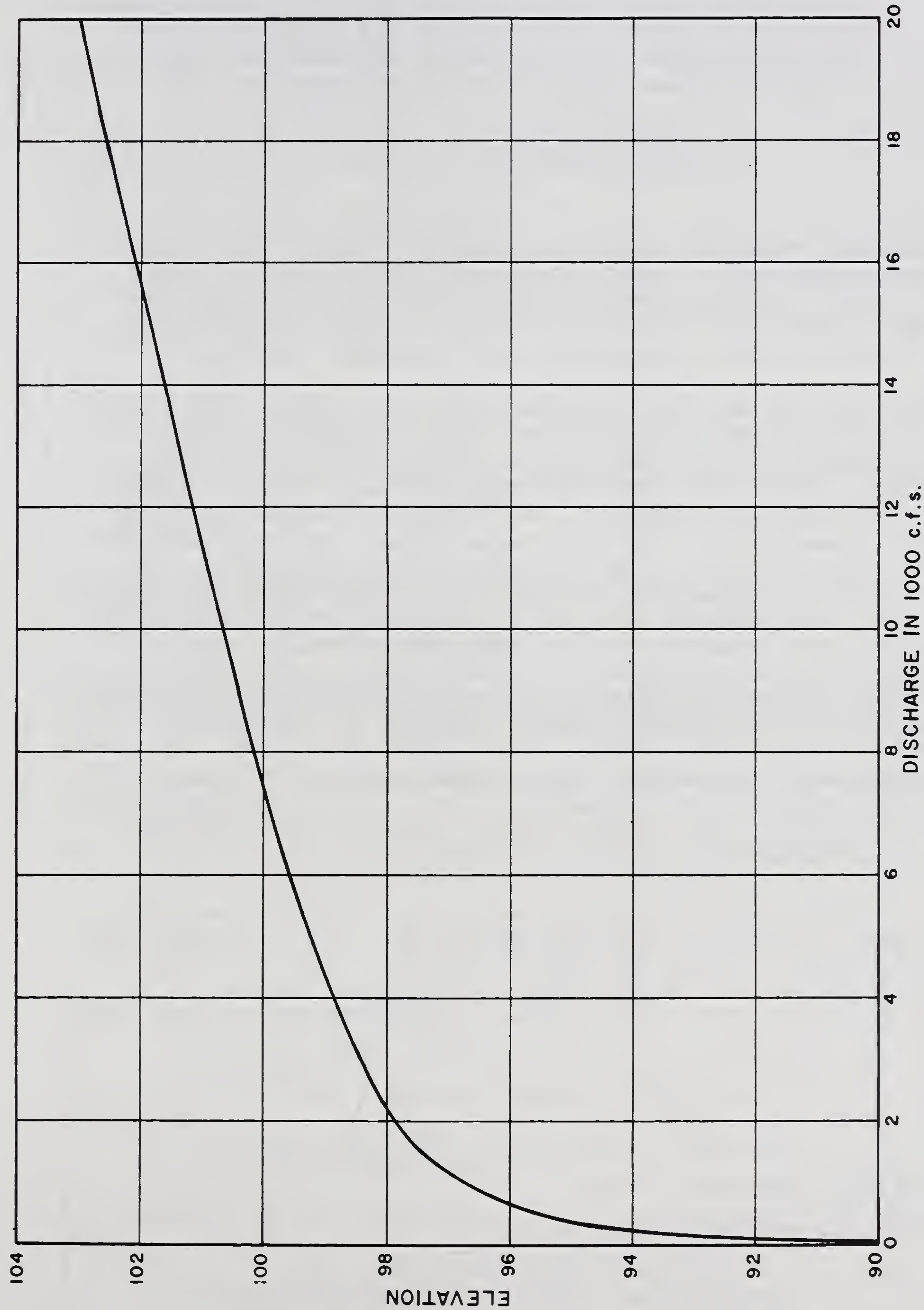


Figure 14-11. Stage discharge section T-1, Example 14-6.

Table 14-5(b) shows computations similar to step 1 through step 11 computing water surface profiles between cross section M-2 on the main stem and T-1 the first cross section on a tributary. Kd values are shown on Figure 14-10. Figure 14-11 was plotted from Table 14-5(b).

Road Crossings

Bridges

In developing the hydraulics of natural streams, bridges of all types and sizes are encountered. These bridges may or may not have a significant effect on the stage discharge relationship in the reach above the bridge. Many of the older bridges were designed without regard to their effect on flooding in the reach upstream from the road crossing.

The Bureau of Public Roads (BPR) in cooperation with Colorado State University initiated a research project with Colorado State University in 1954 which culminated in the investigation of several features of the bridge problem. Included in these investigations was a study of bridge backwater. The laboratory studies, in which hydraulic models served as the principal research tool, have been completed and since then considerable progress has been made in the collection of field data by the U.S. Geological Survey to substantiate the model results and extend the range of application. The procedure developed is explained in the publication "Hydraulics of Bridge Waterways," U. S. Department of Transportation, Federal Highway Administration, Bureau of Public Roads, 1970. This is one method which is recommended by the Soil Conservation Service for use in computing effects of bridges in natural channels and floodplains.

The FHWA document may be obtained from the Superintendent of Documents, U. S. Government Printing Office, Washington, D. C. and it should be included in the working files of any engineer concerned with the effect of bridges on stream hydraulics.

The Bureau of Public Roads (BPR) Method has been formulated by applying the principle of conservation of energy between the point of maximum backwater upstream from the bridge and a point downstream from the bridge at which normal stage has been re-established. The general expression for the computation of backwater upstream from a bridge constricting the flow is:

$$h_1^* = K^* \frac{\alpha_2 V_{n2}^2}{2g} + \left(\frac{\alpha_4 V_4^2}{2g} - \frac{\alpha_1 V_1^2}{2g} \right) \qquad \text{(Eq. 14-23)}$$

where h_1^* = total backwater, in feet

K^* = total backwater coefficient

$\alpha_1, \alpha_2, \alpha_4$ = velocity head energy coefficients at the upstream, constriction, and downstream section.

V_{n2} = average velocity in constriction or $\frac{Q}{A}$ in feet per second.

V_4 = average velocity at section 4 downstream in feet per second.

V_1 = average velocity at section 1 upstream in feet per second.

(For a more detailed explanation of each term and the development of the equation refer to "Hydraulics of Bridge Waterways.")

Equation 14-23 is reasonably valid if the channel in the vicinity of the bridge is essentially straight, the cross sectional area of the stream is fairly uniform, the gradient of the bottom is approximately constant between sections 1 and 4, the flow is free to expand and contract, there is no appreciable scour of the bed in the constriction and the flow is in the subcritical range.

This procedure relates the total backwater effect to the velocity head caused by the constriction times the total backwater coefficient. The total backwater coefficient is comprised of the effect of constriction as measured by the bridge opening coefficient, M, type of bridge abutments, size, shape and orientation of piers, and eccentricity and skew of bridge.

For a detailed discussion of the backwater coefficient and the effect of constriction, abutments, piers, eccentricity and skew of bridges refer to "Hydraulics of Bridge Waterways."

A preliminary analysis may be made to determine the maximum backwater effect of a bridge. If the analysis shows a significant bridge effect then a more detailed procedure should be used. If the analysis shows only a minor effect then the bridge may be eliminated from the backwater computation.

The examples shown in this chapter are based on the approximate equation to compute bridge head losses taken from the BPR report:

$$h^* = K^* \frac{V^2}{2g} \qquad \text{(Eq. 14-24)}$$

where: h^* = total backwater, in feet

K^* = total backwater coefficient

V = average velocity in constriction $\frac{Q}{A}$

A = gross water area in constriction measured below normal stage.

The following data are the minimum needed for estimating the maximum backwater effect of a bridge using Equation 14-24.

1. Total area of bridge opening.

2. Length of bridge opening.

3. Cross section upstream from the bridge a distance approximately equal to the length of the bridge opening.

4. Area of approach section at elevation of the bottom of bridge stringers or at the low point in the road embankment.

5. Width of flood plain in approach section.

6. Estimate of the velocity of unrestricted flow at the elevation of the bottom of the bridge stringers or at the low point in the road embankment.

A preliminary analysis to determine an estimate of the maximum backwater effect of a bridge is shown in Example 14-7. Exhibits 14-2 and 14-3 were developed only for use in making preliminary estimates and should not be used in a more detailed analysis.

Example 14-7.
Estimate the backwater effect of a bridge with 45° wingwalls given the following data: area of bridge = 4100 sq. ft., length of bridge = 400 ft., area of approach = 11850 sq. ft., width of flood plain = 2650 ft., estimated velocity in the natural stream = 2.5 ft./sec.

1. Compute the ratio of the area of the bridge to the area of approach section. From the given data: 4100/11850 = .346

2. Compute the ratio of length of bridge to the width of the flood plain. From the given data: 400/2650 = .151

3. Determine the change in velocity head. Using the results of step 1 (.346) and the estimated velocity in the natural stream (2.5 ft/sec), read the velocity head, h, from Exhibit 14-2. This is the velocity head, $\frac{V^2}{2g}$ in Equation 14-24 and (from Exhibit 14-2) is 0.8 ft.

4. Estimate the constriction ratio, M. Using the results from step 1 (.346) and step 2 (.151) read M = .67 from Exhibit 14-3.

5. Estimate the total backwater coefficient. Using M = .67 from step 4 read from Exhibit 14-4 curve 1, K_b = .6. K_b is the BPR base curve backwater coefficient and for estimating purposes is considered to be the total backwater coefficient, K^*, in Eq. 14-24.

6. Compute the estimated total change in water surface, h*. From Equation 14-24 the total change in water surface is $h^* = K^* \frac{V^2}{2g} = (.6)(.8) = .48$ ft.

If the estimate shows a change in water surface that would have an appreciable effect on the evaluation or level of protection of a plan or the design and construction of proposed structural measures, a more detailed survey and calculation should be made for the bridge and flood in question.

Example 14-8 shows a more detailed solution to the backwater loss using Equation 14-24. In order to use the BPR method it is necessary to develop stage discharge curves for an exit and an approach section assuming no constriction between the two cross sections.

The exit section should be located downstream from the bridge a distance approximately twice the length of the bridge. The approach section should be located upstream from the upper edge of the bridge a distance approximately equal to the length of the bridge.

If the elevation difference between the water surface at the exit section and the approach section prior to computing head loss is relatively small the bridge tailwater may be taken as the elevation of the exit section and the bridge head loss simply added to the water elevation of the approach section. However, if this difference is not small the bridge tailwater should be computed by interpolation of the water elevation at the approach section and exit section and the friction loss from the bridge to the approach section recomputed after the bridge headwater is obtained.

In Example 14-8 it is assumed that all preliminary calculations have been made. The profiles are shown on Figure 14-12a and the stage discharge curve for cross section M-5 is shown on Figure 14-13, Natural Condition.

Example 14-8

Develop stage discharge curves for each of four bridges located at cross section M-4 (Figure 14-4), 300, 400, 500, and 700 feet long (Figure 12c) with 45° wingwalls. The elevation of the bottom of the bridge stringer is 10^3 for each trial bridge length. The main span is 100 feet with the remaining portion of the bridge supported by 24" H-columns on 25 foot centers. Assume the fill is sufficiently high to prevent over topping for the maximum discharge (70000 cfs) studied. It is assumed that water surface profiles have been run for present conditions through section M-5 and that this information is available for use in analyzing the effects of bridge losses.

1. Select a range of discharges that will define the rating curve. For this problem select a range of discharges from 5000 to 70000 cfs for each bridge length and tabulate in column 1 of Table 14-6.

2. Determine present condition elevation for each discharge at the bridge section M-4. For this example water surface profiles have been computed from section M-3 to M-5 without the bridge in place. The results are plotted in Figure 14-12a. From Figure 14-12a read the normal elevation for each discharge at cross section M-4 and tabulate in column 2 of Table 14-6.

3. Compute the elevation vs. gross bridge opening area. The gross area of the bridge is the total area of the bridge opening at a given elevation without regard to the area of

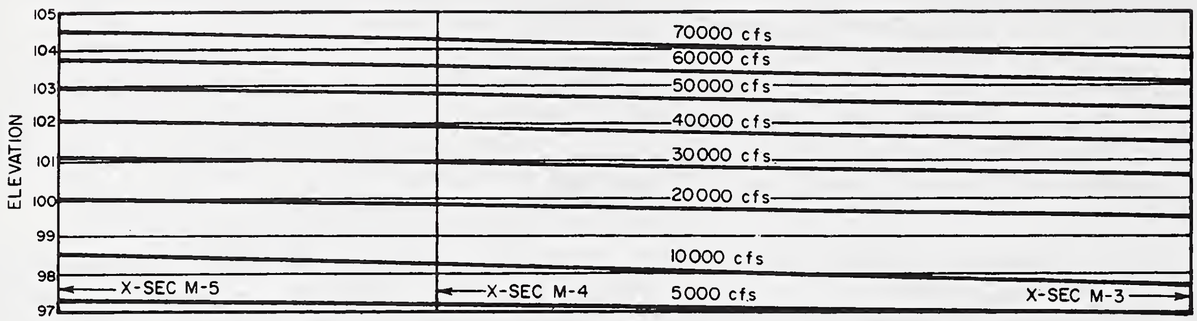


Figure 14-12a. Water surface profile without constriction. Example 14-8.

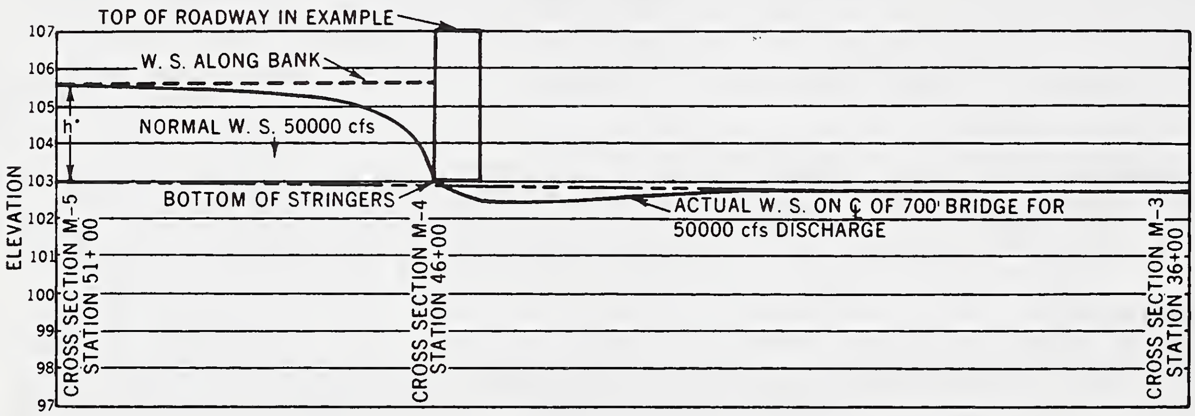


Figure 14-12b. Water surface profile with constriction. Example 14-8.

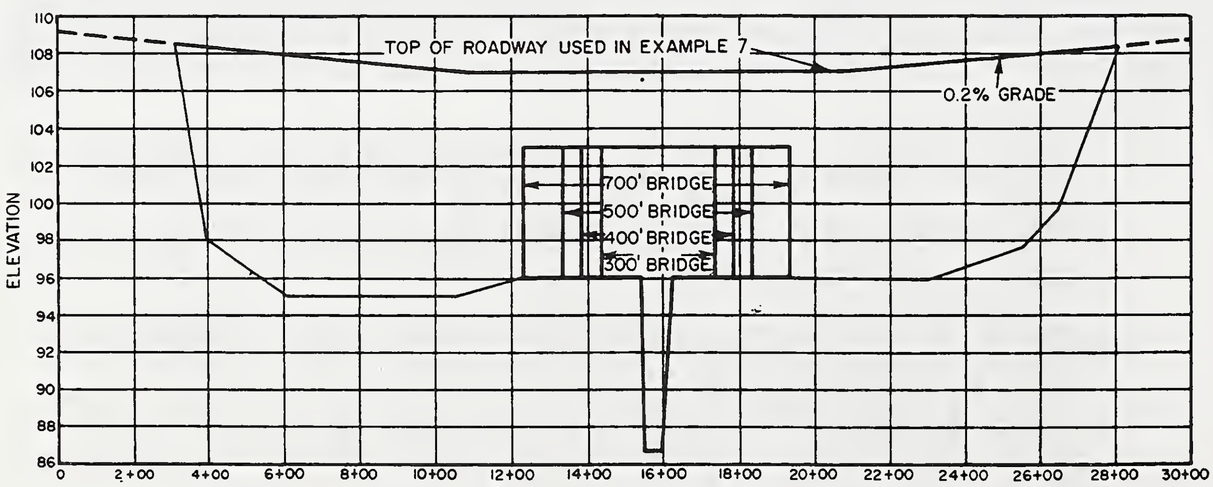


Figure 14-12c. Cross section of road at section M-4, Example 14-8.

Table 14-6. Backwater computations through bridges, Example 14-8.

	Discharge in 1000 cfs	Normal el. @ x-sec M-4	Restricted area A_{n2}	Velocity through bridge openings	Normal el. @ x-sec M-5	M1/	K_b1/	J1/	ΔK_p1/	K*1/	$\frac{V_{n2}^2}{2g}$1/	h*	Elev. with 24" H. col. 25' on cen.
	(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)
300' Bridge 45° Wingwalls	5	97.20	885	5.65	97.35	.470	1.24	.020	.06	1.30	.495	.64	97.99
	10	98.25	1280	7.81	98.55	.350	1.74	.028	.08	1.82	.947	1.72	100.27
	20	99.82	1720	11.63	100.00	.276	2.08	.035	.08	2.16	2.10	4.53	104.53
	30	100.95	2060	14.56	101.15	.243	2.24	.038	.08	2.32	3.29	7.63	108.78
	40	101.90	2340	17.10	102.10	.222	2.34	.040	.08	2.42	4.54	10.99	113.09
	50	102.80	2600	19.23	103.00	.208	2.41	.041	.08	2.49	5.74	14.29	117.29
	60	103.55	2660	22.56	103.75	.183	2.54	.042	.08	2.62	7.90	20.70	124.45
	70	104.25	2660	26.32	104.50	.160	2.66	.042	.07	2.73	10.76	29.37	133.87
400' Bridge 45° Wingwalls	5	97.20	1030	4.85	97.35	.510	1.09	.027	.10	1.19	.365	.43	97.78
	10	98.25	1470	6.80	98.55	.385	1.59	.036	.12	1.71	.718	1.23	99.78
	20	99.82	2070	9.66	100.00	.315	1.90	.043	.12	2.02	1.45	2.93	102.93
	30	100.95	2540	11.81	101.15	.282	2.05	.046	.12	2.17	2.17	4.71	105.86
	40	101.90	2950	13.56	102.10	.265	2.13	.048	.12	2.25	2.86	6.44	108.54
	50	102.80	3300	15.15	103.00	.250	2.21	.049	.12	2.33	3.56	8.29	111.29
	60	103.55	3380	17.72	103.75	.220	2.35	.049	.11	2.46	4.89	12.03	115.78
	70	104.25	3380	20.71	104.50	.192	2.49	.049	.10	2.59	6.66	17.25	121.75
500' Bridge 45° Wingwalls	5	97.20	1160	4.31	97.35	.525	1.03	.032	.13	1.16	.288	.33	97.88
	10	98.25	1670	5.99	98.55	.420	1.44	.042	.15	1.59	.557	.89	99.44
	20	99.82	2550	7.84	100.00	.350	1.74	.049	.16	1.90	.955	1.81	101.81
	30	100.95	3050	9.84	101.15	.325	1.85	.052	.16	2.01	1.50	3.02	104.17
	40	101.90	3520	11.36	102.10	.310	1.92	.054	.16	2.08	2.00	4.16	106.26
	50	102.80	3950	12.66	103.00	.298	1.98	.055	.16	2.14	2.49	5.33	108.33
	60	103.55	4050	14.81	103.75	.262	2.15	.055	.14	2.29	3.41	7.81	111.56
	70	104.25	4050	17.28	104.50	.230	2.30	.055	.13	2.43	4.64	11.28	115.78
700' Bridge 45° Wingwalls	5	97.20	1420	3.52	97.35	.580	0.84	.040	.19	1.03	.192	.20	97.55
	10	98.25	2170	4.61	98.55	.480	1.20	.050	.21	1.41	.330	.47	99.02
	20	99.82	3300	6.06	100.00	.415	1.46	.056	.21	1.67	.570	.95	100.95
	30	100.95	4080	7.35	101.15	.394	1.55	.058	.21	1.76	.839	1.48	102.63
	40	101.90	4750	8.42	102.10	.377	1.62	.059	.21	1.83	1.10	2.01	104.11
	50	102.80	5380	9.29	103.00	.367	1.67	.060	.20	1.87	1.34	2.51	105.47
	60	103.55	5520	10.87	103.75	.325	1.85	.061	.19	2.04	1.83	3.73	107.48
	70	104.25	5520	12.68	104.50	.285	2.04	.061	.17	2.21	2.50	5.53	110.03

1/ These letters and symbols are the same as used in Hydraulics of Bridge Waterways, U. S. Dept. of Transportation, Bureau of Public Roads, 1970. This publication is for sale by Superintendent of Documents.

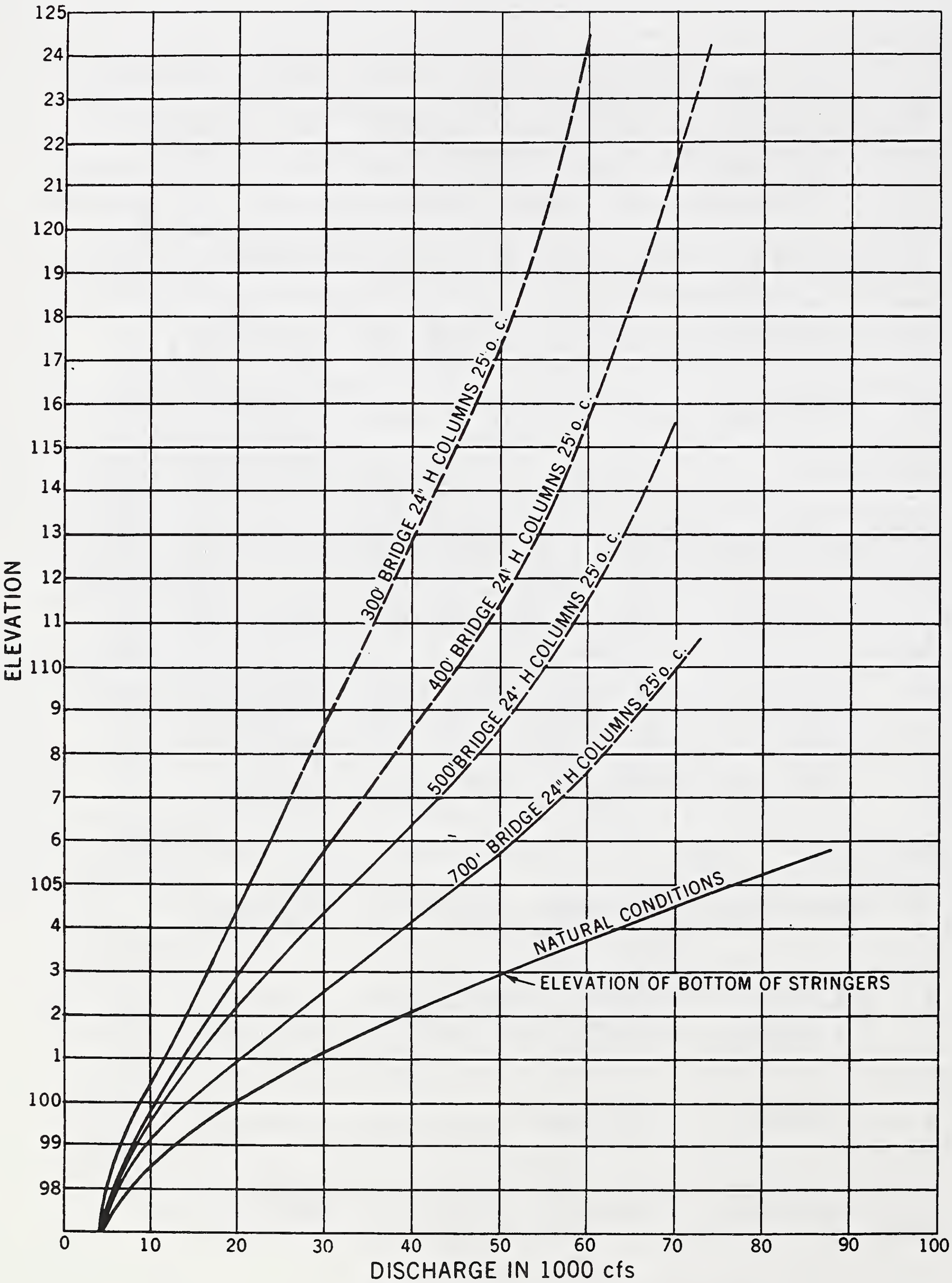


Figure 14-13. Stage discharge without embankment overflow. Section M-5, Example 14-8.

piers. The channel area is 600 ft.^{2} and for the 300 ft. long bridge the gross bridge area is:

Elevation	Bridge Area
96	600
97	900
99	1500
103	2700

Plot the elevation vs. gross bridge opening area as shown in Figure 14-14.

4. Determine the gross area of the bridge opening at each water surface elevation. Using Figure 14-14 read the gross area at each elevation tabulated in column 2 and tabulate in column 3 of Table 14-6.

5. Compute the average velocity through the bridge opening. Divide column 1 by column 3 and tabulate in column 4 of Table 14-5. For the 300 ft. long bridge:

$$V = \frac{Q}{A} = \frac{5000}{885} = 5.65 \text{ ft./sec.}$$

6. Compute the velocity head $(V^2)/2g$. Using the velocities from column 4 compute the velocity head for each discharge and tabulate in column 11 of Table 14-6. For a discharge of 5000 cfs and a bridge length of 300 feet the velocity head is $\frac{(5.65)^2}{(2)(32.2)} =$ .495

7. Determine the elevation for each discharge at section M-5 under natural conditions. Using Figure 14-12a or Figure 14-13 (natural condition curve) read the elevation for each discharge at cross section M-5 and tabulate in column 5 of Table 14-6.

8. Compute M vs. elevation for each bridge size. M is computed as outlined in "Hydraulics of Bridge Waterways." It is computed as the ratio of that portion of the discharge at the upstream section computed for a width equal to the length of the bridge to the total discharge of the channel system. If Q_b is the discharge at the upstream section computed for a flood plain or channel width equal to the length of the bridge and Q_a and Q_c is the remaining discharge on either side of Q_b then $M = \frac{Q_b}{Q_a+Q_b+Q_c} = \frac{Q_b}{Q}$.

   The bridge opening ratio, M, is most easily explained in terms of discharges, but it is usually determined from conveyance relations. Since conveyance (Kd) is proportional to discharge, assuming all subsections to have the same slope, M can be expressed also as:

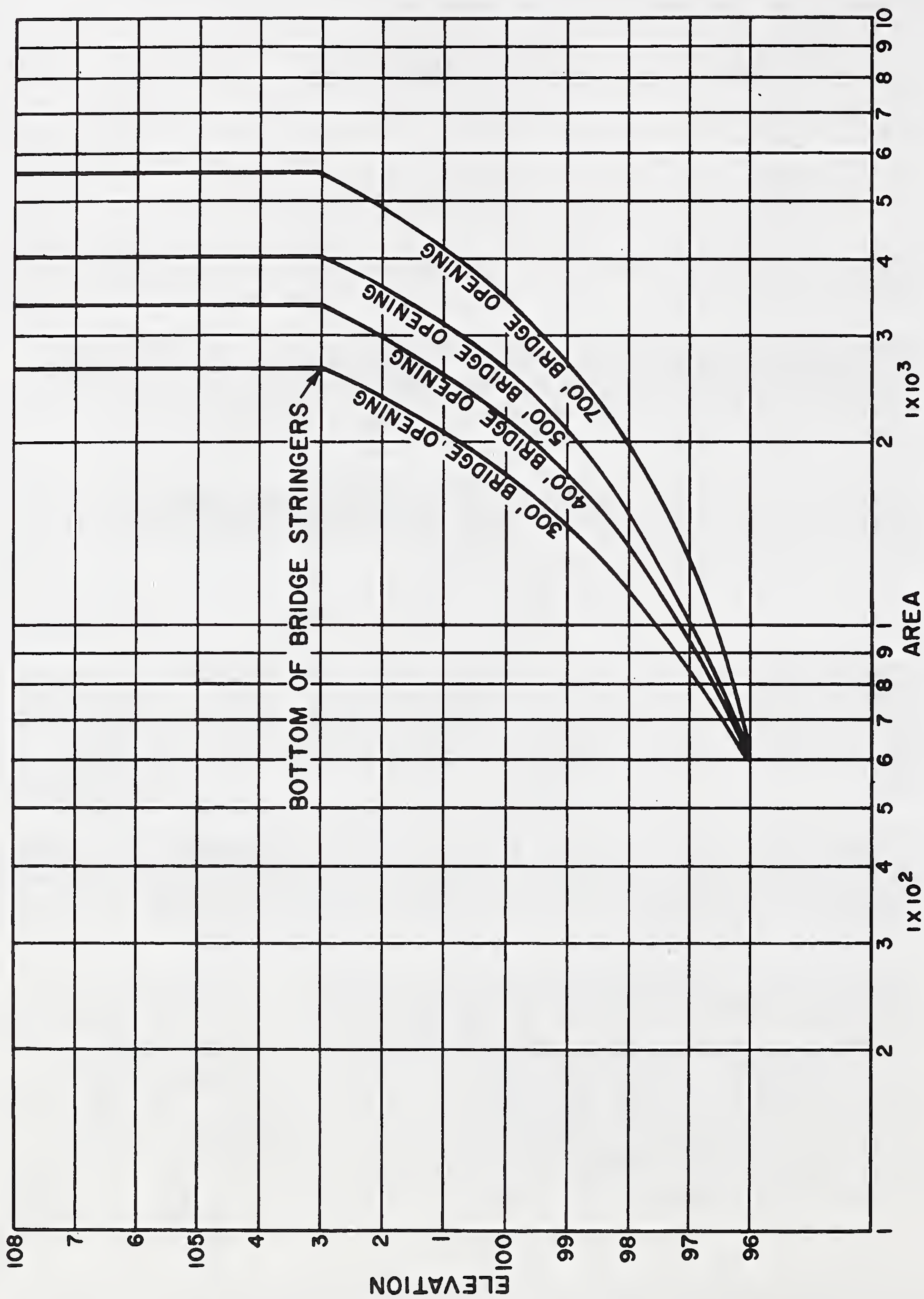


Figure 14-14. Bridge opening areas, Example 14-8.

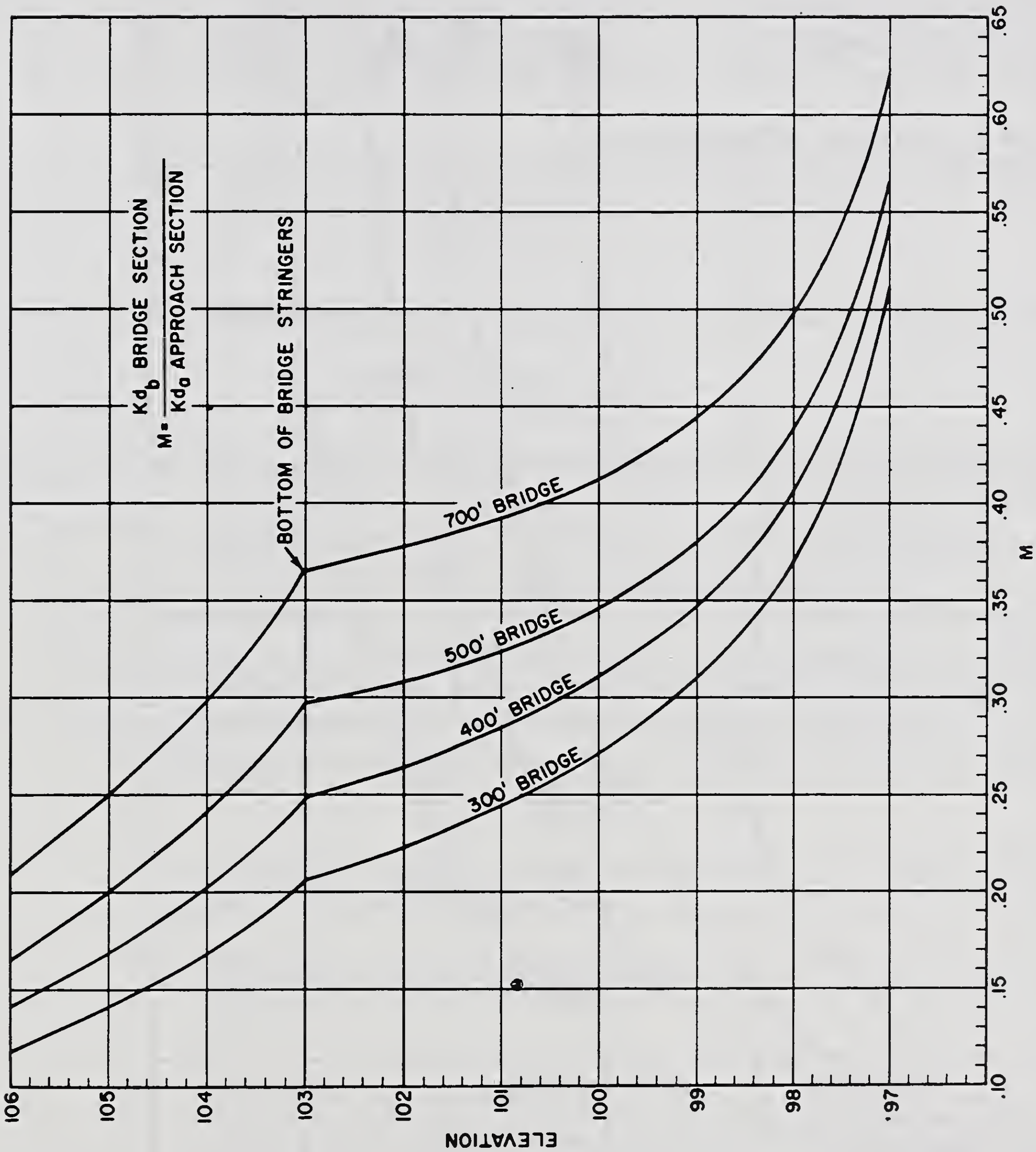


Figure 14-15. M values for bridge, Example 14-8.

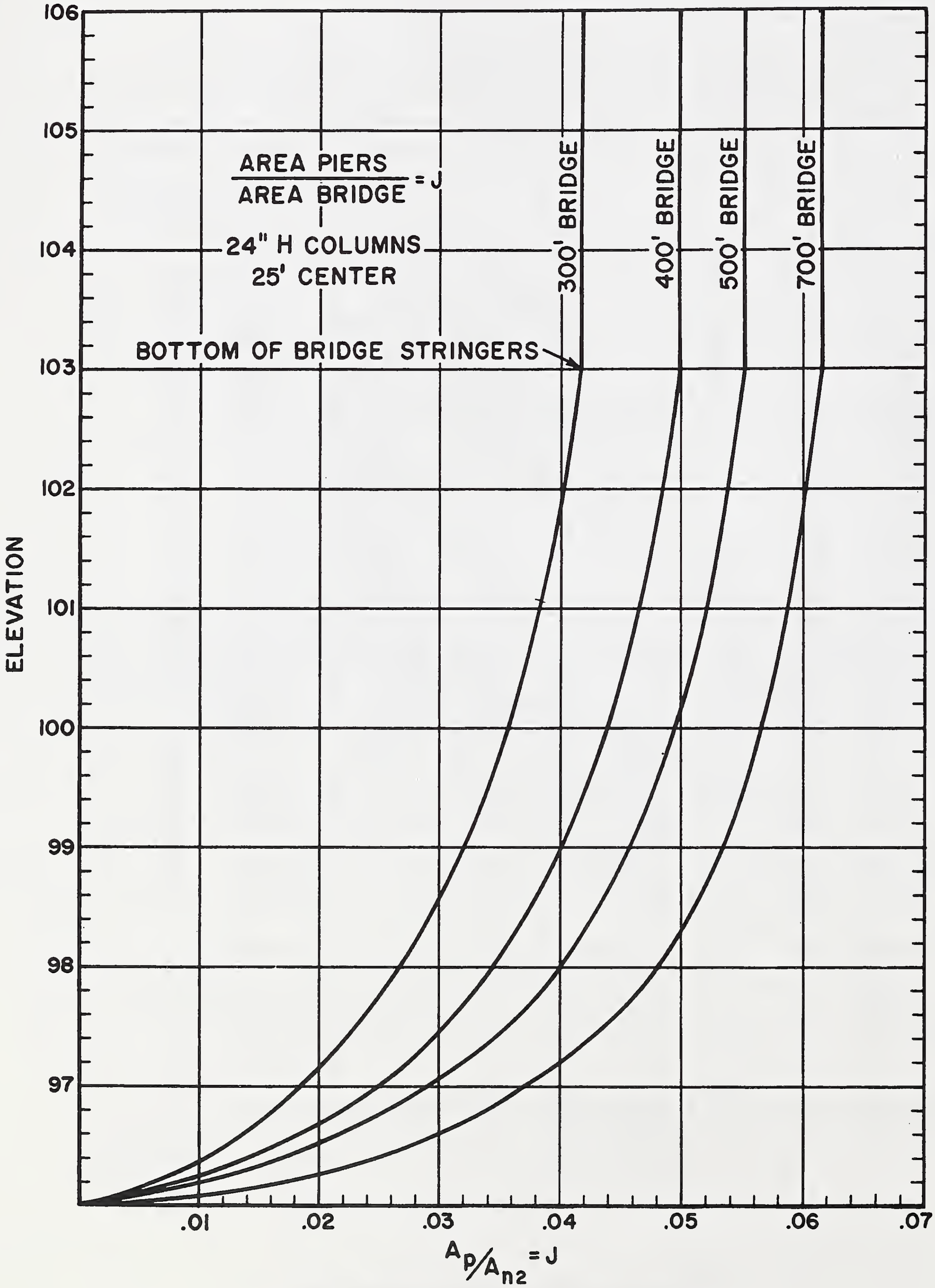


Figure 14-16. J values for bridge, Example 14-8.

$$M = \frac{Kd_b}{Kd_a+Kd_b+Kd_c} = \frac{Kd_b}{Kd}$$

The approach section information is not shown for this example.

Plot M vs. elevation for each bridge size as shown in Figure 14-15.

9. Read M for each elevation. Using Figure 14-15 prepared in step 8 read M for each elevation in column 2 and tabulate in column 6 of Table 14-6.

10. Determine the base backwater coefficient K_b. Using M from step 9, read K_b Exhibit 14-4 for bridges having 45° wingwalls and tabulate in column 7 of Table 14-6.

11. Compute the area of pier/area of bridge vs. elevation.

$$\frac{\text{area of piers}}{\text{area of bridge}} = \frac{A_p}{A_{n2}} = J$$

For the 300' bridge the piers are located in an area 200' wide. (300' - 100' clear span = 200'). The piers are on 25 foot centers and are 2 feet wide. Within the 200 foot width the piers will occupy $\frac{(200)}{(25)}$ (2) = 16 feet.

At an elevation of 103 the piers will occupy an area 25 feet wide by 7 feet deep (103-96 = 7 feet). From Figure 14-14 the gross area of the bridge opening is 2700 feet.

Then: $$\frac{A_p}{A_{n2}} = \frac{(16)(7)}{2700} = .41$$

Compute and plot A_p/A_{n2} vs. elevation for each bridge length as shown in Figure 14-16.

12. Determine J for each elevation. Read J from Figure 16-16 for each elevation in column 2 and tabulate in column 8 of Table 14-6.

13. Determine the incremental backwater coefficient ΔK_p.

Using J from step 12 read ΔK from the appropriate curve (for this example curve 1) from Exhibit 14-5a. Using M from step 9 read σ from the appropriate curve (curve 1) from Exhibit 14-5b. Multiply ΔK by σ and tabulate as ΔK_p in column 9 of Table 14-6.

for 5000 cfs and a 300' bridge:

$$\Delta K = .105 \qquad \sigma = .59$$

$$\Delta K_p = \Delta K\sigma = (.105)\ (.59) = .06$$

14. Determine the total backwater coefficient K*. Add columns 7 and 9 and tabulate as K* in column 10. This is the total backwater coefficient for the bridge that will be considered for this example. If there are other losses that appear to be significant, the user should follow the procedure shown in the BPR report for computing their effects.

15. Determine the total change in water surface h*. Multiply column 10 by column 11 and tabulate in column 12. From Eq. 14-24:

$$h^* = K^* \frac{V^2}{2g}$$

for 5000 cfs and a 300 foot bridge with piers:

$$h^* = (1.30)\ (.495) = .64 \text{ feet}$$

If the example did not include piers or if the effect of eliminating the piers are desired the h* could be determined by multiplying column 7 by column 11.

for 5000 cfs and a 300 foot bridge without piers:

$$h^* = (1.24)\ (.495) = .61 \text{ feet}$$

16. Determine the elevation with bridge losses. Add column 5 and column 12 and tabulate in column 13. Column 13 is plotted on Figure 14-13 which shows the stage discharge curve for cross section M-5, assuming the fill to be high enough to force all of the 70,000 cfs discharge through the bridge opening.

* *

Full bridge flow

The analysis of flood flows past existing bridges involves flows which submerge all or a part of the bridge girders. When this condition occurs the computation of the head loss through the bridge must allow for the losses imposed by the girders. This may be accomplished in several ways.

One method is to continue using the BPR method but hold the bridge flow area and Kd constant for all elevations above the bridge girder. Example 14-8 uses this procedure. (See Figure 14-14).

Another approach commonly taken is to compute the flow through the bridge opening by the orifice flow equation.

$$q = CA\sqrt{2g\Delta h} \qquad \text{(Eq. 14-25)}$$

where q = discharge, in cfs
Δh = the difference in water surface elevation between headwater and tailwater, in feet
A = flow area of bridge opening, in square feet
g = acceleration of gravity
C = coefficient of discharge

In estimating C, if conditions are such that flow approaches the bridge opening with relatively low turbulence, the appropriate value of C is about 0.90. In the majority of cases C probably is in the 0.70 to 0.90 range. For very poor conditions (much turbulence), it may be as low as 0.40 to 0.50. In judging a given case, consider the following.

(1) Whether the abutments are square-cornered or shaped so as to reduce turbulence
(2) the number and shape of piers
(3) the degree of skew
(4) the number and spacing of pile bents since closely-spaced bents increase turbulence
(5) the existence of trees, drift, or other types of obstruction at the bridge or in the approach reach.

Using a C value of 0.8 has given approximately the same results as the BPR method for Example 14-7. However, the corresponding C value varied with discharge.

* *

Overtopping of bridge embankment

When the fill of a bridge is overtopped the total discharge at the bridge section is equal to the discharge through the bridge opening plus the discharge over the embankment. A reliable estimate of the effect of the bridge constriction on stages upstream under these conditions is difficult to obtain.

A generally accepted procedure to use in analyzing flows over embankments is to consider the embankment as acting as a broad crested weir. The broad crested weir equation is:

$$Q = CLH_e^{3/2} \qquad \text{(Eq. 14-26)}$$

where L = length of weir, in feet
H_e = energy head which is comprised of the velocity head at the upstream section plus the depth of flow over the weir, in feet
C = a coefficient

The following approximate ranges of C values for flows over embankments are recommended for use in Eq. 14-26. For road and highway fills, C = 2.5 to 2.8; for single-track railroad fills, C = 2.2 to 2.5; for double-track railroad fills, C = 1.9 to 2.2.

Equation 14-26 was developed for use in rectangular weir sections. Since road profiles encountered in the field seldom represent rectangular sections

it becomes difficult to determine the weir length to use. Many approaches have been formulated to approximate this length. One approach suggests measuring the top width at the maximum depth of flow over the road and computing $H_e = d_c + \frac{A}{2T}$ for each depth.

Another method suggests measuring the weir length from the cross section at an elevation equal to 5/6 of h above the low point on the embankment.

A method suggested for use in this chapter substitutes the flow area A for the weir length and flow depth over the weir in Eq. 14-26.

Then: $$Q = C'Ah^{1/2} \qquad \text{(Eq. 14-27)}$$

where:
- A = flow area over the embankment at a given depth, h, in square feet
- h = flow depth measured from the low point on the embankment, in feet
- C' = coefficient which accounts for the velocity of approach.

The coefficient C' can be computed by equating Equations 14-26 and 14-27 and solving for C'.

$$C' = C\left(\frac{1}{\frac{\text{depth}}{\text{depth+velocity head}}}\right)^{3/2} \qquad \text{(Eq. 14-28)}$$

In Eq. 14-28 the depth is measured from the low point on the embankment of the bridge section and the velocity head is computed at the upstream section for the same elevation water is flowing over the embankment. The approach velocity may be approximated by V = Q/A where Q is the total discharge and A is the total flow area at the upstream section for the given elevation. In cases where the approach velocity is sufficiently small C' will equal C and no correction for velocity head will be needed to use Equation 14-27.

The free discharge over the road computed using Eq. 14-27 must be modified when the tailwater elevation downstream is great enough to submerge the embankment of the bridge section. The modification to the free discharge, Q_f, is made by computing a submergence ratio, H_2/H_1, where H_2 and H_1 are the depths of water downstream and upstream, respectively, above the low point on the embankment. A submergence factor, R, is read from Figure 3-4, NEH-11, Drop Spillways, and the submerged discharge is computed as $Q_s = RQ_f$. Then the total discharge at the bridge section is equal to the discharge through the bridge opening plus the submerged discharge over the embankment.

Example 14-9 shows the use of Eq. 14-27 and Eq. 14-28 in computing flows over embankments using a trial and error procedure to determine C'.

Example 14-9.
Develop a stage discharge curve for the overflow section of the highway analyzed in Example 14-8 (see Figure 14-12c) for the bridge opening of 300 feet. The top of embankment is at elevation 107. Assume a C value of 2.7.

1. Select a range of elevations that will define the rating curve over the road. Tabulate in column 1 of Table 14-7. The low point on the road is at elevation 107.

2. Compute the depth of flow, h, over the road. For each elevation listed in column 1 compute h and list in column 2 of Table 14-7.

3. Compute $h^{1/2}$. Tabulate in column 3 of Table 14-7.

4. Compute the flow area, A, over the road. For each elevation listed in column 1 compute the area over the road and tabulate in column 4 of Table 14-7.

Steps 5 through 11 are used to calculate the modified coefficient, C' to account for the approach velocity head. If it is determined that no modification to the coefficient C is required these steps may be omitted.

5. Compute the flow area at the upstream section. For each elevation listed in column 1 compute the total area at the upstream section and tabulate in column 5 of Table 14-7. The flow area can be obtained from the Kd computations at the upstream section or computed directly from the surveyed cross section.

6. Determine the discharge through the bridge. For the elevation in column 1 read the discharge through the bridge opening previously computed using bridge loss equations and tabulate in column 6 of Table 14-7.

7. Estimate the discharge over the road. Tabulate in column 7 of Table 14-7.

8. List the total estimated discharge going past the bridge section. Sum columns 6 and 7 and tabulate in column 8 of Table 14-7.

9. Compute the average velocity at the upstream section. The velocity can be estimated by using the total upstream area from column 5 and the estimated discharge from column 8 for the elevations listed in column 1 in the equation V = Q/A. For example for elevation 107.5:

$$V = \frac{28250 \text{ ft}^3/\text{sec}}{26700 \text{ ft}^2} = 1.06 \text{ ft/sec.}$$

Tabulate the velocity in column 9 of Table 14-7.

Table 14-7. Stage discharge over roadway at cross section M-4 without submergence. Example 14-9.

Elevation	h ft	$h^{1/2}$	A over road ft^2	A up-stream ft^2	Q through bridge cfs	Q est. over road cfs	Q est. total cfs	V ft/sec	$\frac{V^2}{2g}$	C'	Q over road cfs	Q total cfs
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)
107.0	0.	0.	0.	25500.	26000.	0	26000	1.02	.016	0	0	26000
107.5	.5	.707	625	26700.	27000.	1300	28300	1.06	.0175	2.85	1300	28300
108.0	1.0	1.000	1500	28000.	28000.	4300.	32300	1.15	.021	2.79	4200	
						4200.	32200	1.15	.021	2.79	4200	32200
108.5	1.5	1.225	2525.	29200.	29300.	8500.	37800	1.30	.027	2.76	8500	37900.
109.0	2.0	1.414	4000.	30400.	30300.	12000	42000	1.38	.030	2.76	15600	
						15600	45900	1.51	.035	2.76	15600	45900.
110.0	3.0	1.730	7500.	32800.	32800.	35000	67800	2.06	.066	2.79	36200	
						36200	69000	2.10	.068	2.79	36200	69000

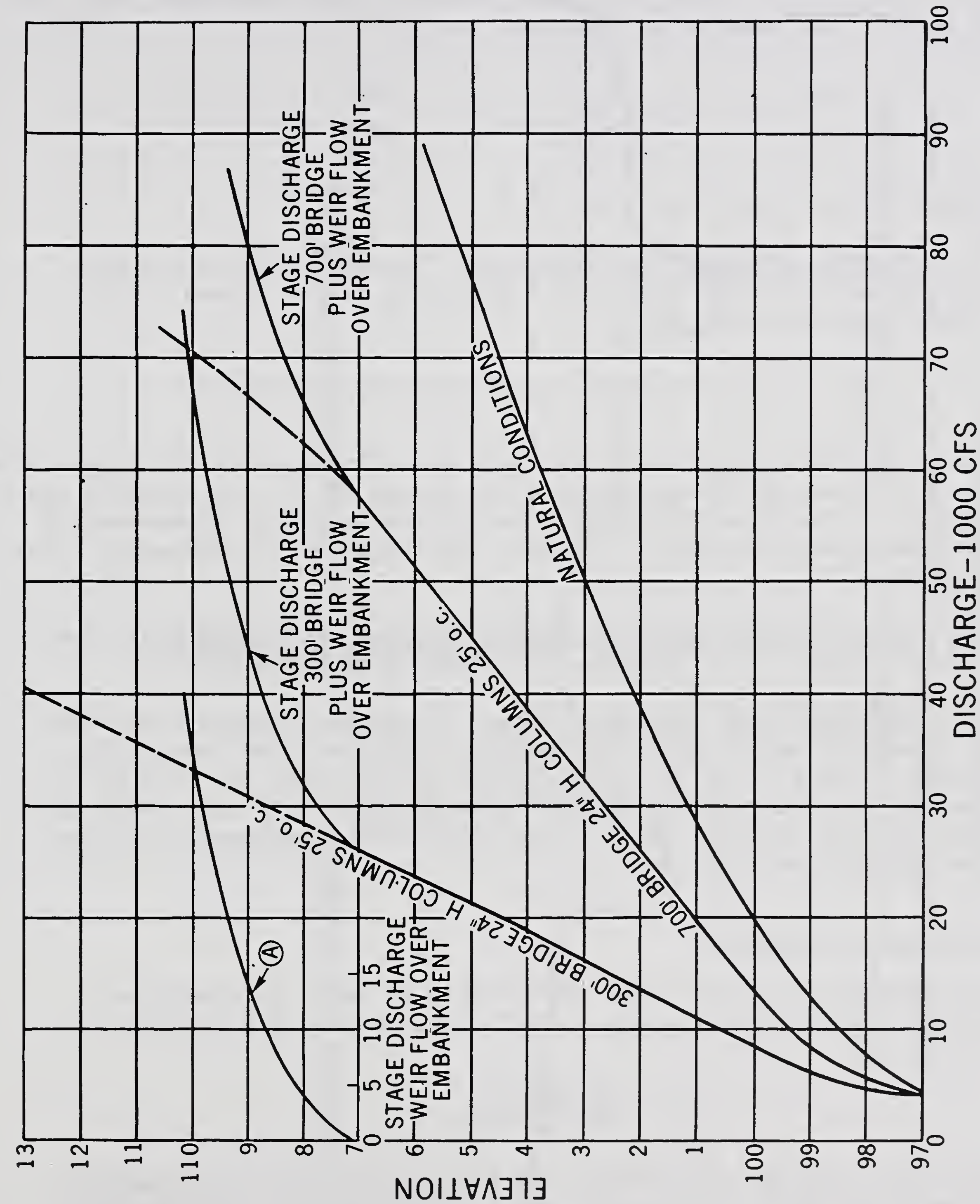


Figure 14-17. Stage discharge with embankment overflow, section M-5, Example 14-9.

10. Compute the velocity head. Using the velocity from column 9 compute $V^2/2g$ and tabulate in column 10 of Table 14-7.

11. Compute C'. Using equation 14-28 and data from Table 14-7 compute C'. For example at elevation 107.5:

$$C' = 2.7 \frac{1}{\left(\frac{.5}{.5+.0175}\right)^{3/2}} = \frac{2.7}{(.966)^{3/2}} = 2.85$$

List C' in column 11 in Table 14-7.

12. Compute discharge over the road. Using equation 14-27 and data from Table 14-7 compute the discharge over the road. For example at elevation 107.5:

$$Q = C'Ah^{1/2} = 2.85(625)(.707) = 1260 \text{ cfs}$$

Round to 1300 cfs and list in column 12. Compare this discharge value to the estimated discharge listed in column 7. If the computed discharge is less than or greater than the estimated discharge modify the estimated discharge in column 7 and recompute C' following steps 8 through 12.

13. List the total discharge going past the bridge section. Sum columns 6 and 12 and Tabulate in column 13 of Table 14-7.

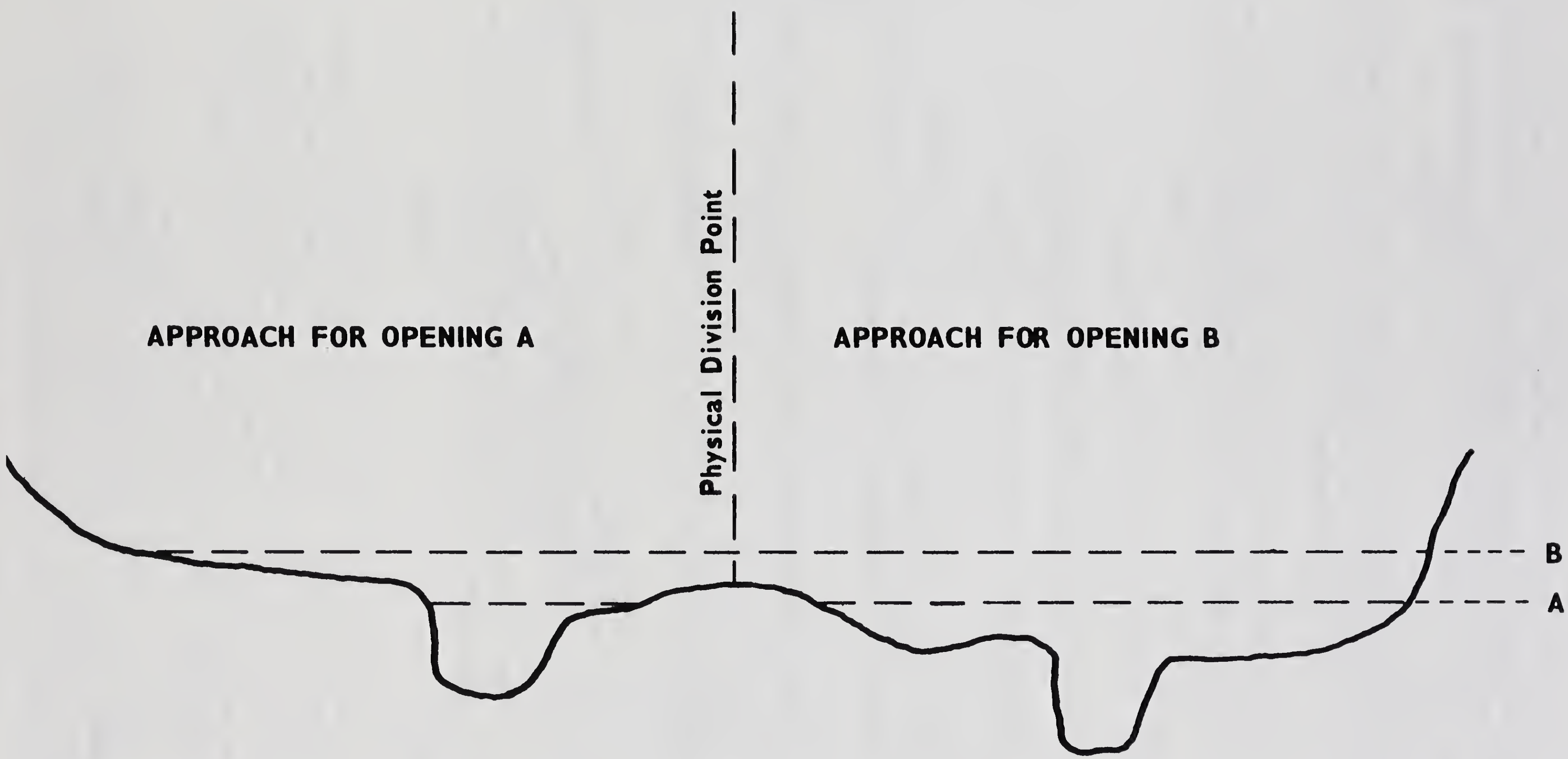
14. Plot the stage discharge curve. Using the computations shown in columns 1 and 13 of Table 14-7 plot the elevation versus discharge. The portion of the discharge flowing over the road (column 12) and the total discharge curve is shown in Figure 14-17 for the 300 foot bridge. This is the total stage discharge curve for the approach section (M-5).

Multiple bridge openings

Multiple openings in roads occur quite often and must be considered differently from single openings. The M ratio in the BPR procedure is defined as:

$$\frac{\text{Kd Bridge}}{\text{Kd Approach}}$$

When multiple openings are present the proper ratio must be assigned to each opening and then the capacity computed accordingly. If the flow is divided on the approach, the porblem is then one of divided flow with single openings in each channel. In many cases the flow is not divided



When water elevation is at A approaches act as directed by the physical division point.

When water elevation is at B approaches act according to the ratio of KD's of openings.

Figure 14-18. Approach section for a bridge opening.

for overbank flows. In these cases the headwater elevation must be considered to be the same elevation for each opening and the solution becomes trial and error until the head losses are equal for each opening and the sum of the flows equals the desired total.

The approaches are divided as shown in Figure 14-18. When the headwater is below the physical dividing point as illustrated by Level A then the M ratio is computed as in a single opening.

When the headwater is above the physical dividing point cross flow can occur. When this occurs the approach used to compute the M ratio and J is as follows:

1. Compute the Kd value for each bridge opening.

2. Compute the Kd value for the total approach section.

3. Proportion the approach Kd value for each opening by the relationship:

$$Kd\ appr_x = \frac{Kd_{bridge_x}}{Kd_{bridge_1} \ldots . Kd_{bridge_2} \ldots . + Kd_{bridge_n}} \times \text{total approach Kd.}$$

4. Compute M as before using the Kd value computed in step 3 for the approach.

5. Compute the approach area contributing to this opening by the relationship:

$$Area\ appr_x = \frac{Kd_{bridge_x}}{Kd_{bridge_1} \ldots . + Kd_{bridge_2} \ldots . Kd_{bridge_n}} \times \text{total approach area}$$

6. Compute J as before using the area computed in step 5 for the approach area.

Culverts

Culverts of all types and sizes are encountered when computing stage discharge curves in natural streams. These culverts may or may not have a significant effect on the development of a watershed work plan. However, in many cases they present a problem in evaluating a plan and must be analyzed to determine if an acceptable plan can be installed without enlarging or replacing the existing culvert.

The Bureau of Public Roads has developed procedures based on research data for use in designing culverts. This document, Hydraulic Charts for the Selection of Highway Culverts, Hydraulic Engineering Circular No. 5, December 1965, is available from the Superintendent of Documents, Washington, D. C.

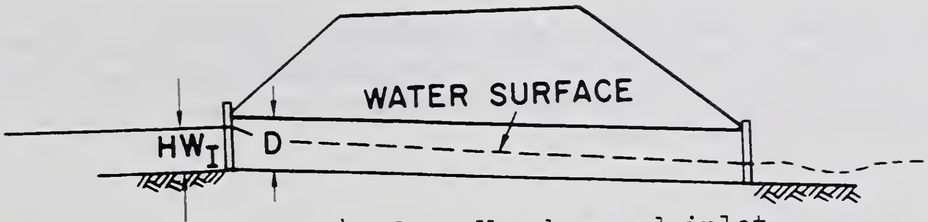


Figure 14-19a. Unsubmerged inlet

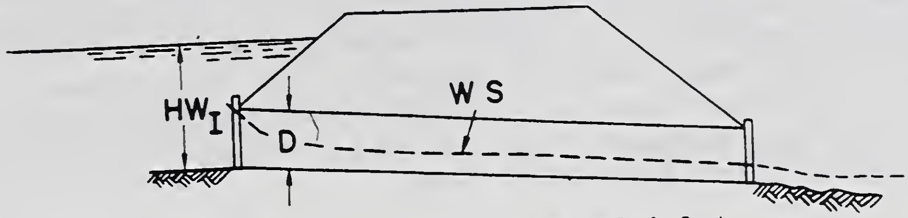


Figure 14-19b. Submerged inlet

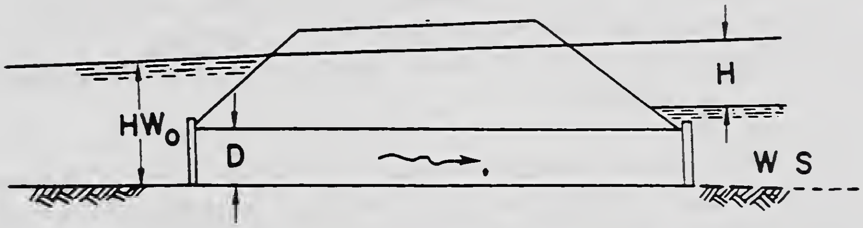


Figure 14-19c. Submerged outlet

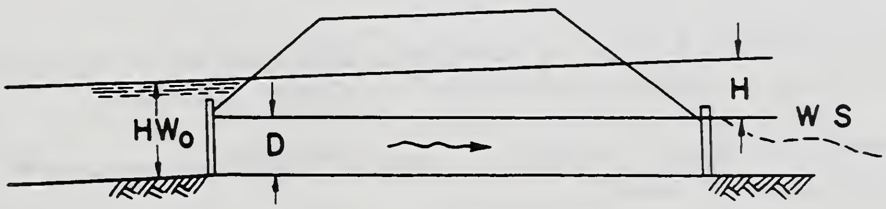


Figure 14-19d. Outlet flowing full

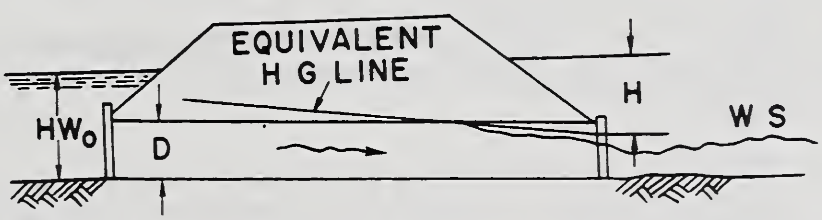


Figure 14-19e. Pipe full part way

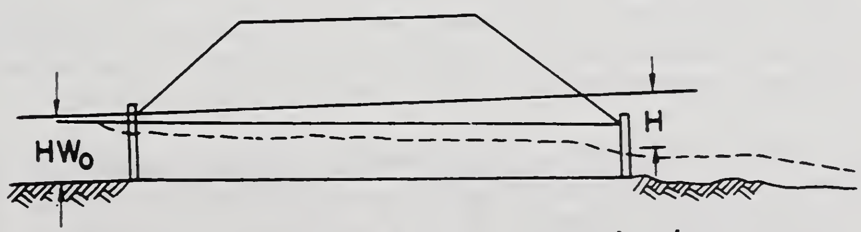


Figure 14-19f. Open flow through pipe

Culverts of various types, installed under different conditions, were studied in order to develop procedures to determine the backwater effect for the two flow conditions: 1) culverts flowing with inlet control; 2) culverts flowing with outlet control.

Inlet Control

Inlet control means that the capacity of the culvert is controlled at the culvert entrance by the depth of headwater (HW_I) and the entrance geometry of the culvert including the barrel shape and cross sectional area and the type of inlet edge, shape of headwall, and other losses. With inlet control the entrance acts as an orifice and the barrel of the culvert is not subjected to pressure flow. Figure 14.19a and 14.19b show sketches of two types of inlet controlled flow.

The nomographs shown on Exhibits 14-6 through 14-10 were developed from research data by the Division of Hydraulic Research, Bureau of Public Roads research data. They have been checked against actual measurements made by USGS with favorable results.

Types of Inlets. - The following descriptions are taken from "Electronic Computer Program for Hydraulic Analysis of Circular Culverts" Bureau of Public Roads, February 1969. Some of the types of inlets are illustrated in Figure 14-20.

a. Tapered - This inlet is a type of improved entrance with can be made of concrete or metal. Shapes are shown in Figure 14-20a.

b. Bevel A and Bevel B - These bevels, a type of improved entrance, can be formed of concrete or metal.

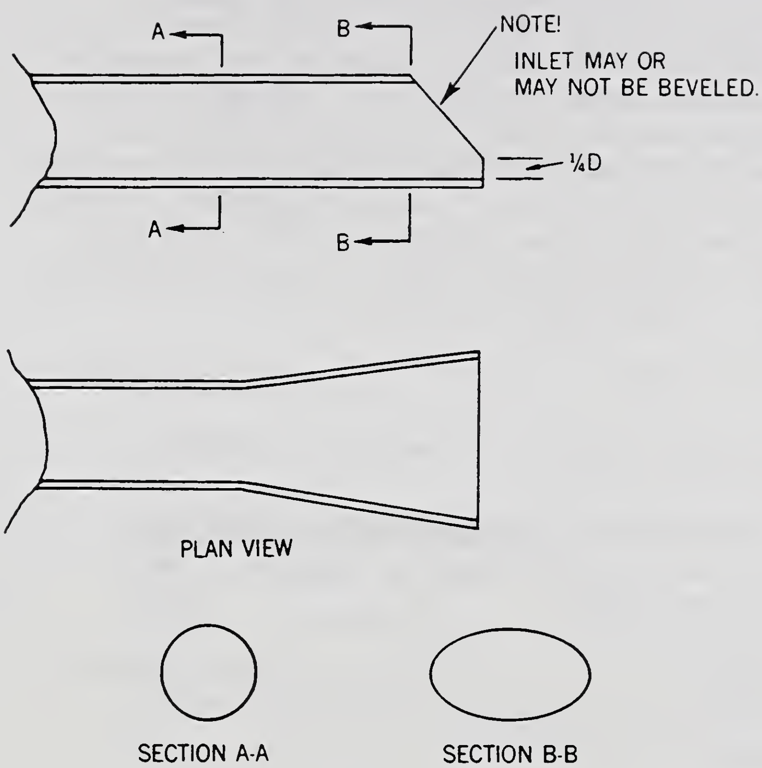
c. Angled wingwall - Similar to headwall but at an angle with culvert.

d. Projecting - The culvert barrel extends from the embankment. The transverse section at the inlet is perpendicular to the longitudinal axis of the culvert.

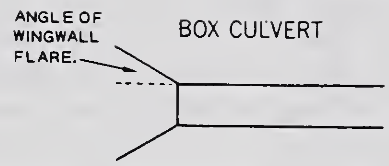
e. Headwall - A headwall is a concrete or metal structure placed around the entrance of the culvert. Headwalls considered are those giving a flush or square edge with the outside edge of the culvert barrel. No distinction is made for wingwalls with skewed alignment.

f. Mitered - The end of the culvert barrel is on a miter or slope to conform with the fill slope. All degrees of miter are treated alike since research data on this type of inlet are limited. Headwater is measured from the culvert invert midway into the mitered section.

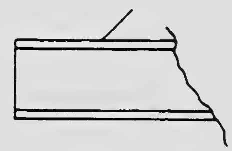
g. End section - This section is the common prefabricated end made of either concrete or metal and placed on the inlet or outlet ends of a culvert. The closed portion of the section, if present, is not tapered. (Not illustrated)



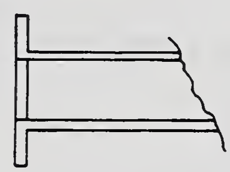
Tapered inlet



Angled wing wall



Projecting



Headwall

BEVEL	$\frac{b}{D}$	$\frac{a}{D}$	$\frac{c}{D}$	$\frac{d}{D}$
A	0.042	0.063	0.042	0.083
B	0.083	0.125	0.042	0.125

Bevel A and Bevel B

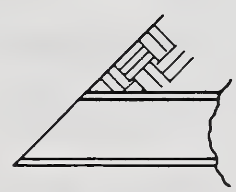


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Figure 14-20. Types of culvert inlets.

h. Grooved edge - The bell or socket end of a standard concrete pipe is an example of this entrance. (Not illustrated)

Outlet Control

Culverts flowing with outlet control can flow with the culvert barrel full or part full for part of the barrel length or for all of it. Figures 14-19c, 14-19d, 14-19e, and 14-19f show the various types of outlet control flow. The equation and graphs for solving the equation give accurate results for the first three conditions. For the fourth condition shown in Figure 14-19f, the accuracy decreases as the head decreases. The head H, Figure 14-19c and 14-19d, or the energy required to pass a given discharge through the culvert flowing in outlet control with the barrel flowing full throughout its length consists of three major parts: 1) velocity head H_v, 2) entrance loss H_e, and 3) friction loss H_f, all expressed in feet. From Figure 14-21a:

$$H = H_v + H_e + H_f \qquad \text{(Eq. 14-29)}$$

$H_v = \frac{V^2}{2g}$ when V is the average velocity in the culvert barrel.

H_e = entrance loss which depends on the geometry of the inlet. The loss is expressed as a coefficient K_e (Exhibit 14-21) times the barrel velocity head.

$$H_e = K_e \frac{V^2}{2g} \qquad \text{(Eq. 14-30)}$$

H_f = friction loss in barrel

$$H_f = \frac{29n^2 L}{R^{1.33}} \times \frac{V^2}{2g} \qquad \text{(Eq. 14-31)}$$

n = Mannings friction factor
L = length of culvert barrel (ft)
V = velocity in culvert barrel (ft/sec)
g = acceleration of gravity (ft/sec^2)
R = hydraulic radius (ft)

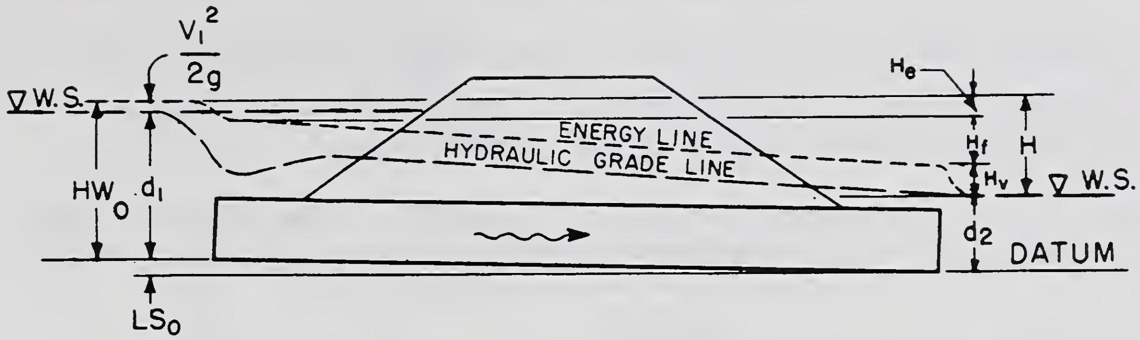
Substituting in Equation 14-23:

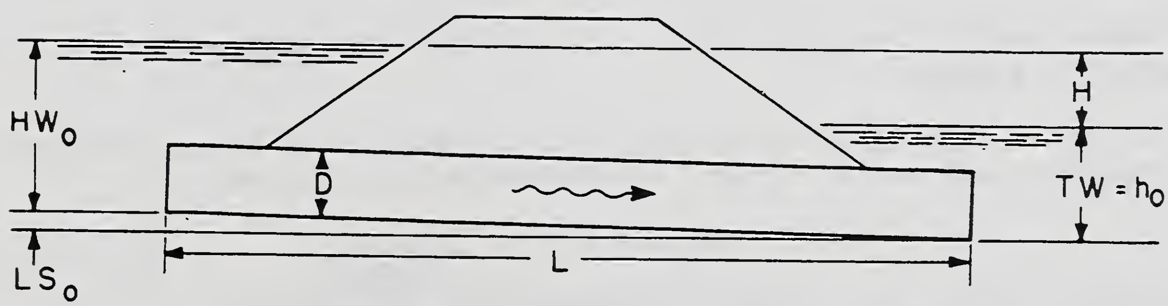
$$H = \left(1 + K_e + \frac{29n^2 L}{R^{1.33}}\right) \frac{V^2}{2g} \qquad \text{(Eq. 14-32)}$$

Figure 14-21a shows the terms of Eq. 14-29, the hydraulic gradeline, the energy gradeline, and the headwater depth HW_o.

The expression for H is derived by equating the total energy upstream from the culvert to the energy just inside the culvert outlet.

$$H = d_1 + \frac{V_1^2}{2g} + LS_o - d_2 = H_v + H_e + H_f \qquad \text{(Eq. 14-33)}$$





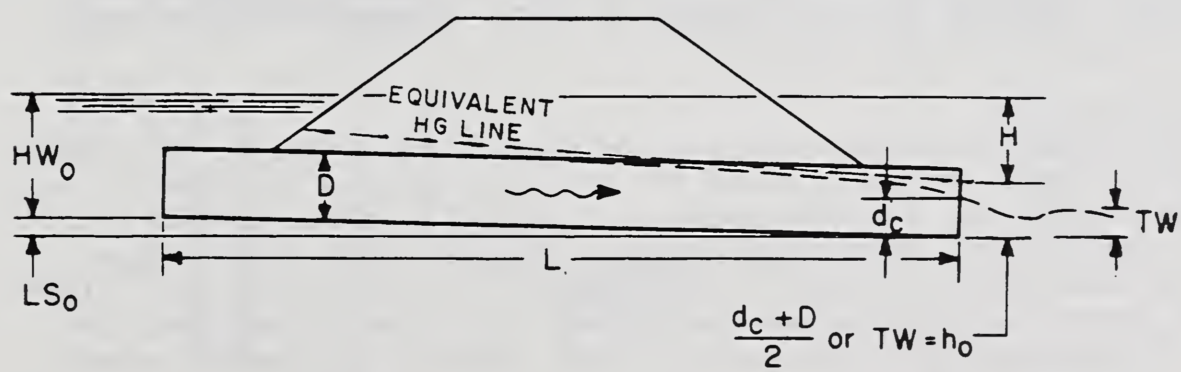


Figure 14-21. Elements of culvert flow.

From Figure 14-21a:

$$HW_o = H + d_2 - LS_o \qquad \text{(Eq. 14-34)}$$

If the velocity head in the approach section $(\frac{V_1^2}{2g})$ is low it can be ignored and HW_o is considered to be the difference between the water surface and the invert of the culvert inlet.

The depth, d_2, for culverts flowing full is equal to the culvert height Figure 14-19d, or the tailwater depth (TW) whichever is greater, Figure 14-21b.

The hydraulic gradeline for culverts flowing with the barrel part full for part of the barrel length passes through a point where the water breaks with the top of the culvert and if extended as a straight line will pass through the plane of the outlet end of the culvert at a point above the critical depth. This point is approximately halfway between d_c and the crown of the culvert, or equal to $\frac{d_c + D}{2}$. The depth d_2 or h_o (see Figure 14-21c) for this type of flow is equal to $\frac{d_c + D}{2}$ or TW whichever is greater.

With the above definition of d_2 which will be designated as h_o, an equation common to all outlet control conditions can be written:

$$HW_o = H + h_o - LS_o \qquad \text{(Eq. 14-35)}$$

This equation was used to develop the nomographs shown on Exhibits 14-11 through 14-15 which can be used to develop stage discharge curves for the approach section to culverts flowing with outlet control.

Exhibit 14-16 shows d_c for discharge per foot of width for rectangular sections. Exhibits 14-17 to 14-20 show d_c for discharges for various non-rectangular culvert sections.

Example 14-10
Develop a stage discharge curve for cross section T-4 (Figure 14-4) showing the backwater effect of eight 16' x 8' concrete box culverts for each of three conditions: 1) inlet control, 2) outlet control, present channel, and 3) outlet control, improved channel. Figure 14-22a shows a cross section along the centerline of the roadway at cross section T-3. Figure 14-22b shows a section through the roadway with water surface profiles prior to and after the construction of the culverts and roadway embankment.

The culvert headwalls are parallel to the embankment with no wingwalls, and the entrance is square on three edges.

The following are given in this example: a stage discharge curve for cross section T-2, present condition and with proposed channel improvement

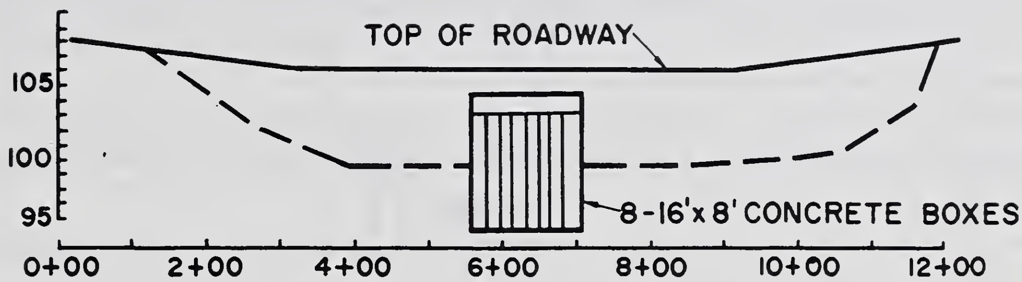


Figure 14-22a. Cross section T-3.

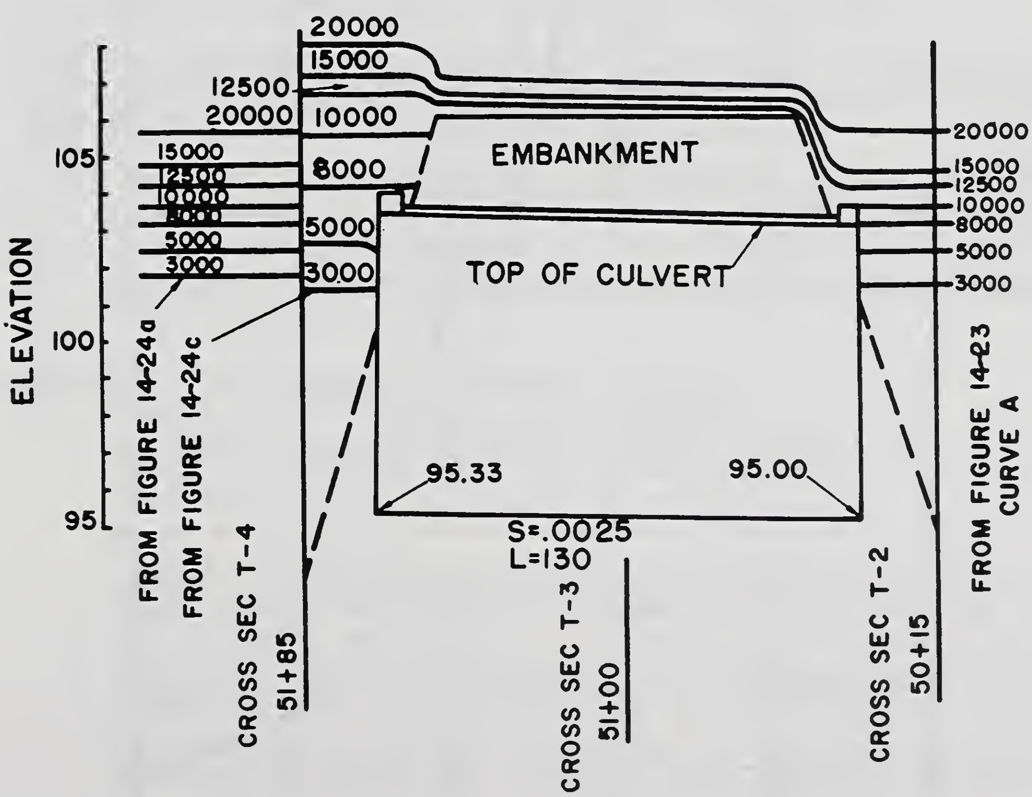


Figure 14-22b. Profile through culvert, Example 14-10.

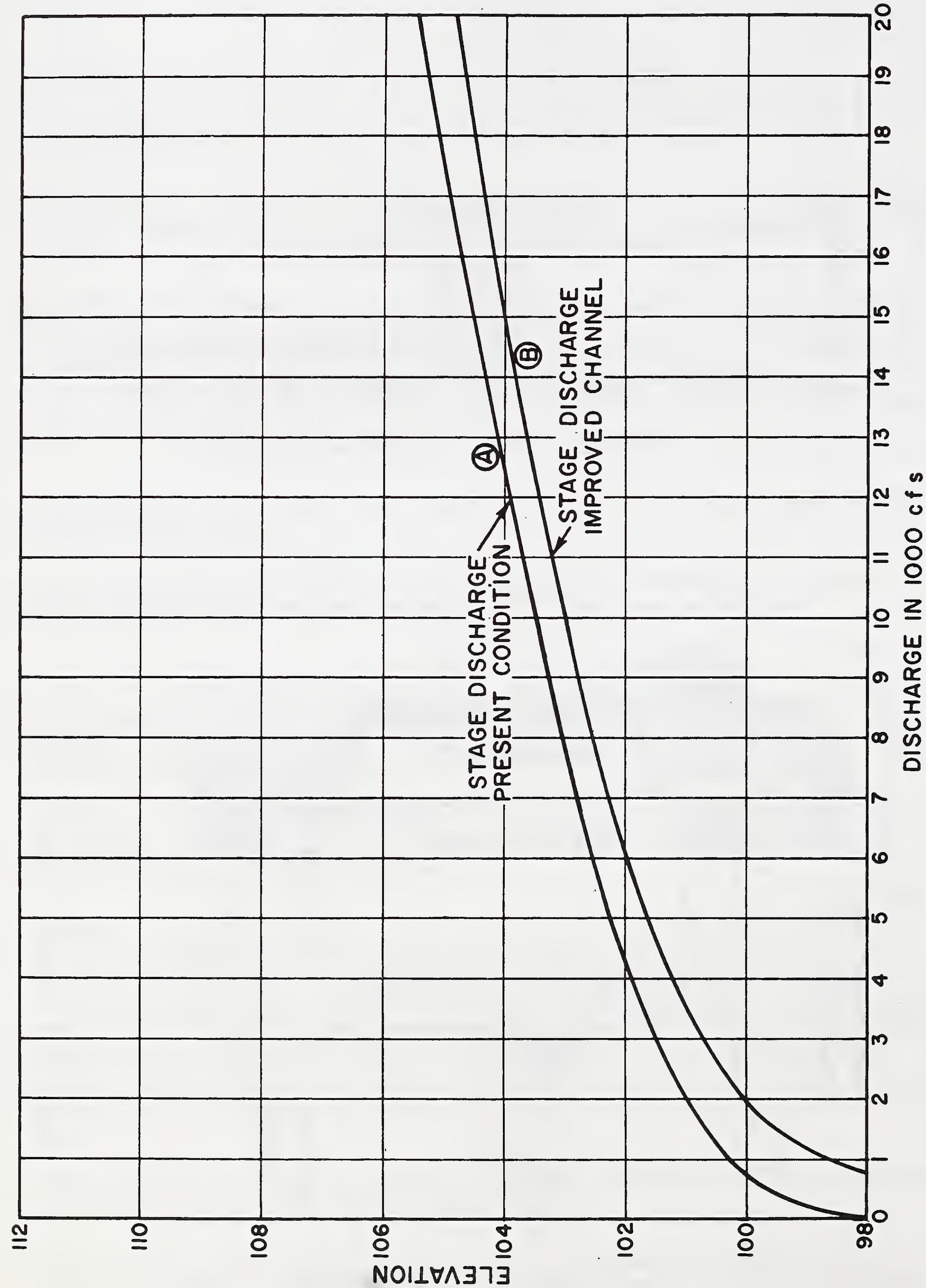


Figure 14-23. Stage discharge exit section T-2, Example 14-10.

(Figure 14-23, curves A and B). Also given is a stage discharge curve for cross section T-4 disregarding the effect of the culverts and roadway fill (Figure 14-24a).

Condition 1--Inlet Control

1. Select a range of discharges sufficient to define the new stage discharge curve. Tabulate in column 1 of Table 14-8.

2. Determine the discharge for each culvert. Divide the discharges in column 1 by the number of culverts (8) and tabulate in column 2 of Table 14-8.

3. Determine the discharge per foot of width. Divide the discharges in column 2 by the width of each culvert (16 feet) and tabulate in column 3 of Table 14-8.

4. Compute $\frac{HW}{D}$. Using the nomograph, Exhibit 14-6, read HW/D for each discharge per foot of width in column 3 and tabulate in column 4 of Table 14-8. Referring to Exhibit 14-6 project a line from the depth of culvert (8 feet) through the discharge per foot of width (line q/B) to the first HW/D line, then horizontal to line (3), which is the HW/D for the type of culvert in this example.

5. Compute HW. Multiply column 4 by the depth of the culvert (8 feet) and tabulate in column 5 of Table 14-8.

6. Add the invert elevation at the entrance to the culvert (elev. 95.33) to column 5. Tabulate in column 6 of Table 14-8.

7. Plot the stage discharge curve assuming inlet control. Plot column 1 and column 6 of Table 14-8 as the stage discharge curves for cross section T-4 (see Figure 14-24b curve A). This assumes inlet control with the road sufficiently high to prevent over topping.

Condition 2--Outlet Control, Present Channel

1. Compute the entrance loss coefficient, K_e. Read $K_e = 0.5$ from Exhibit 14-21 for the type of headwall and entrance to box culvert and tabulate in column 7 of Table 14-8.

2. Compute the head loss, H, for the concrete box culvert flowing full. Using the nomograph on Exhibit 14-11, draw a line from L = 130 feet on the $K_e = 0.5$ scale to the cross sectional area scale, 16' x 8' = 128 square feet, and establish a point on the turning line. Draw a line from the discharge (q) line for each of the discharges shown in column 2, through the turning point to the head (H) line. Tabulate H in column 8 of Table 14-8.

Table 14-8. Headwater computations for eight 16' x 8' concrete box culverts, headwalls parallel to embankment (no wingwalls), square edged on three sides, Example 14-10.

Total Discharge q	Discharge for Each Culv.	Discharge per foot of Width	Inlet Control			Outlet Control, Present Channel								Outlet Control, Improved Channel	
			$\frac{HW}{D}$	HW	HW_I 1/	K_e	H	d_c	$\frac{d_c + D}{2}$	h_o 2/ Elev.	TW Elev.	LS_o	HW_o 3/ Elev.	TW Elev.	HW_o Elev.
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)
3000	375	23.5	0.55	4.40	* 4/	0.5	0.22	2.6	5.30	100.30	101.4	0.33	* 4/	100.7	* 4/
5000	625	39.1	0.77	6.15	* 4/	0.5	0.60 5/	3.6	5.80	100.80	102.3	0.33	102.57	101.6	101.87
8000	1000	62.5	1.08	8.65	103.98 6/	0.5	1.55	4.9	6.45	101.45	103.0	0.33	104.22 6/	102.6	103.82 6/
10000	1250	78.0	1.31	10.46	105.79	0.5	2.50	5.7	6.85	101.85	103.5	0.33	105.67	103.0	105.17
12500	1565	98.0	1.61	12.88	108.21	0.5	3.90	6.7	7.35	102.35	104.0	0.33	107.57	103.6	107.17
15000	1875	117.0	2.01 7/	16.08	111.41	0.5	5.60	7.5	7.75	102.75	104.5	0.33	109.77	104.0	109.27
20000	2500	156.3	--	--	--	0.5	10.00	9.0 8/	8.00 9/	103.00	105.5	0.33	115.77	104.8	114.47

1/ $HW_I = HW + 95.33$ (invert elevation at entrance end of culvert = 95.33).

2/ $h_o = \frac{d_c + D}{2} + 95.00$ (invert elevation at outlet end of culvert = 95.00).

3/ $HW_o = H + TW - LS_o$ or $H + h_o - LS_o$, whichever is greater.

4/ Tailwater elevation is higher than the computed elevation and open channel flow exists.

5/ See example on Exhibit 14-11.

6/ Note: with channel improvement the control switches from outlet to inlet between 5000 and 8000 cfs.

7/ See example on Exhibit 14-6.

8/ If $d_c \geq D$, the outlet always controls.

9/ $\frac{d_c + D}{2}$ cannot exceed D.

3. Compute the critical depth, dc, for each discharge per foot of width. Using Exhibit 14-16, read d_c for each discharge per foot of width shown in column 3 and tabulate in column 9 of Table 14-8.

4. Compute $\frac{d_c + D}{2}$. Tabulate in column 10 of Table 14-8

   Note: $\frac{d_c + D}{2}$ cannot exceed D.

5. Compute h_o. Add the invert elevation of the outlet end of the culvert (elev. 95.00) to $\frac{d_c + D}{2}$ and tabulate as h_o in column 11 of Table 14-8.

6. Compute the TW elevation for each discharge in column 1. Using Figure 14-23, curve A, read the elevation for each discharge in column 1 and tabulate as TW elevation in column 12 of Table 14-8.

7. Compute the difference in elevation of the inlet and outlet inverts of the culverts. Multiply L x S_o = 130 x .0025 = 0.33 and tabulate in column 13 of Table 14-8.

8. Compute the water surface elevation, HW_o, assuming outlet control. Add values in column 8 to the larger of column 11 or column 12 minus column 13 and tabulate as HW_o in column 14 of Table 14-8.

9. Plot the stage discharge curve assuming outlet control. Plot column 1 and column 14 on Figure 14-24c curve A assuming outlet control with the roadway sufficiently high to prevent over topping.

Condition 3-- Outlet Control, Improved Channel.

1. Compute the tailwater elevation at the culvert for the improved channel condition.
   Using Figure 14-23, curve B, read the elevation for each discharge in column 1 and tabulate as TW elevation in column 15 of Table 14-8.

2. Compute the elevation assuming outlet control, improved channel. Add column 8 plus the layer of column 15 or column 15 minus column 13 and tabulate in column 16 of Table 14-8.

3. Plot the stage discharge curve assuming outlet control with improved channel. Plot column 1 and column 16 on Figure 14-24d, curve A, as the stage discharge curve for cross section T-4 assuming outlet control with improved channel and the roadway sufficiently high to prevent over topping.

Condition for flow over roadway.
Assume the approach velocity head for this example is negligable and the coefficient C will equal C^1 used in Eq. 14-26. If the velocity head is significant and a correction to the coefficient C is desired by using Eq. 14-27 follow steps 5 through 9 of Example 14-9.

1. Select a range of elevations that will define the rating curve over the road. Tabulate in column 1 of Table 14-9. The low point on the road is at elevation 106.

2. Compute the depth of flow, H, over the road. For each elevation in column 1 compute H and list in column 2 of Table 14-9.

3. Compute $H^{1/2}$. Tabulate in column 3 of Table 14-9.

4. Compute the flow area, A, over the road. For each elevation listed in column 1 compute the area over the road and tabulate in column 4 of Table 14-9.

5. Determine coefficient, C. Assume C = 2.7 for this example and assume $C = C^1$. Tabulate C^1 in column 5 of Table 14-9.

6. Compute the discharge over the roadway using Eq. 14-26.

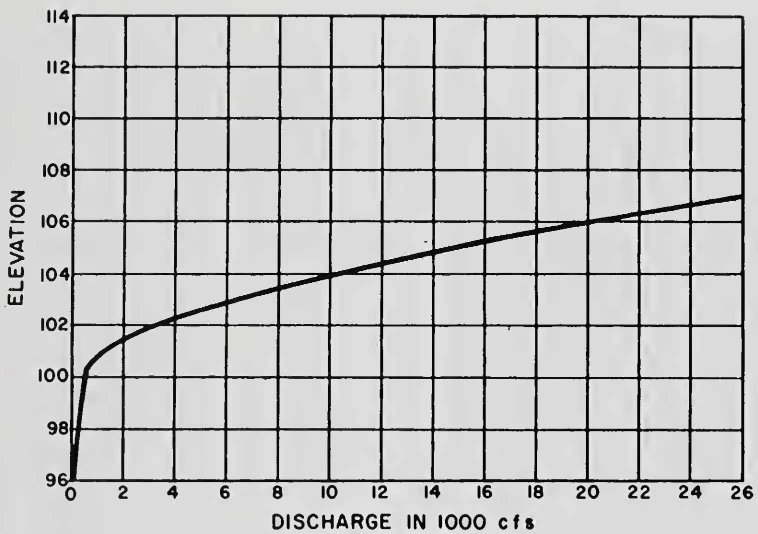
7. Plot the stage discharge curve. Using the computations shown in Table 14-9 plot column 1 and column 6 shown on Figure 14-24b, c, and d as curve B.

8. Graphically combine curves A and B on Figures 14-24b, c and d to form the stage discharge curve for the culverts and weir flow over the roadway.

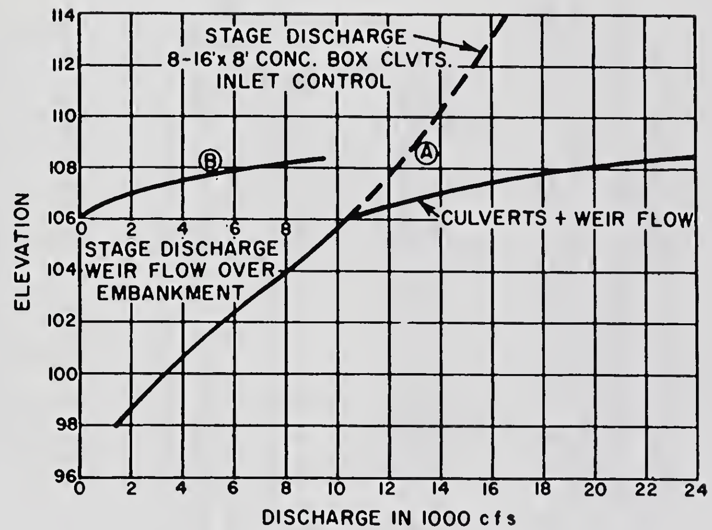
Table 14-9. Stage discharge over roadway at cross section T-3, Figure 14-4. Example 14-10.

Elevation	H	$H^{1/2}$	A	C^1	q
(1)	(2)	(3)	(4)	(5)	(6)
106.	0.	0.	0	2.7	0
106.5	.5	.707	340	2.7	650
107.	1.0	1.	750	2.7	2020
107.5	1.5	1.225	1230	2.7	4070
108	2.0	1.414	1790	2.7	6830

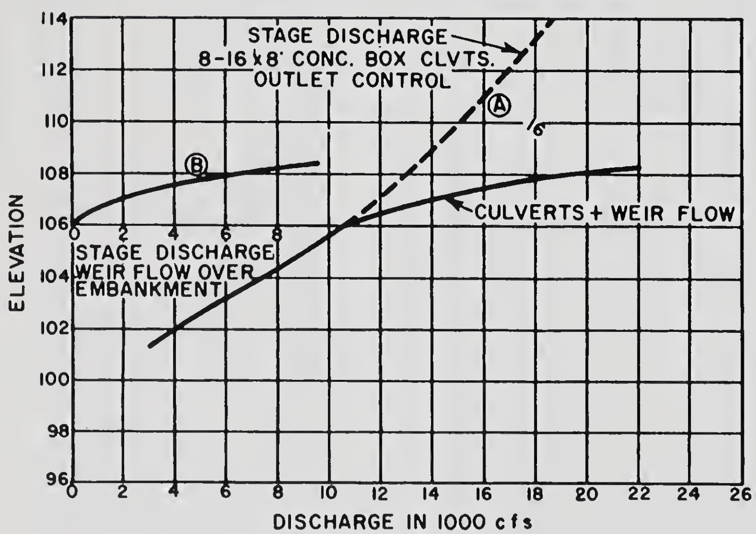
Each of the 3 flow conditions were computed independent of each other. The flow condition that actually controls is that which requires the greater upstream elevation for the discharge being considered. By comparing elevations for the same discharge for the 3 conditions tabulated on Table 14-8 and plotted on Figure 14-24 b, c and d the type of control at any given discharge can be determined. It may be advantageous to plot all the curves on one graph to better define points of intersection.



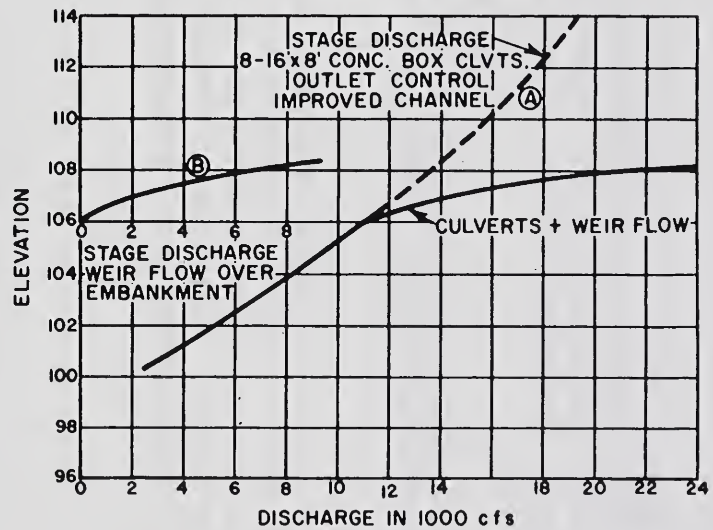
(a) assuming no roadway in place.



(b) inlet control



(c) outlet control.



(d) improved channel-outlet control

Figure 14-24. Rating curves cross section T-4, Example 14-10.

Under the old channel conditions it can be determined that open channel flow conditions exist for discharges less than about 4000 cfs, outlet control governs between about 4000 and 9000 cfs and inlet control governs for discharges greater than 9000 cfs.

Under new channel conditions open channel flow exists for discharges less than 3800 cfs, outlet control governs for discharges between 3800 and 7300 cfs and inlet control governs for discharges greater than 7300 cfs. Also, in both cases, discharges greater than 10,200 cfs flow will occur over the road embankment.

If the actual profile for discharges occurring under open channel flow conditions is desired water surface profiles should be run through the culverts.

It can also be seen from Figure 14-24a and 14-24b that by constructing the highway with 8 - 16' x 8' concrete box culverts elevations upstream will increase over present conditions for discharges greater than 5000 cfs. for improved outlet conditions upstream elevations will not be increased above present conditions until a discharge of 7200 cfs occurs.

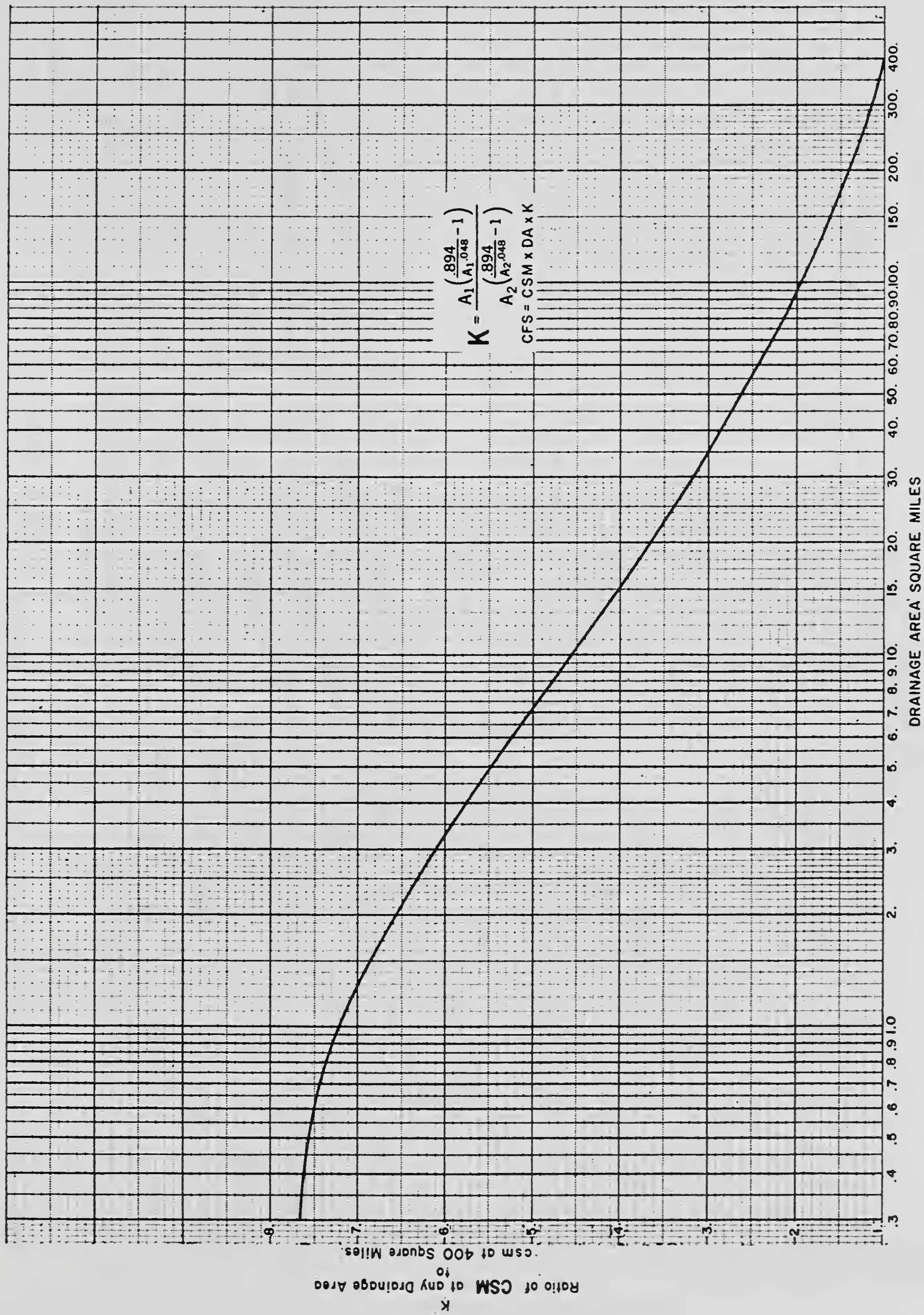


Exhibit 14-1. K values for converting CSM to CFS.

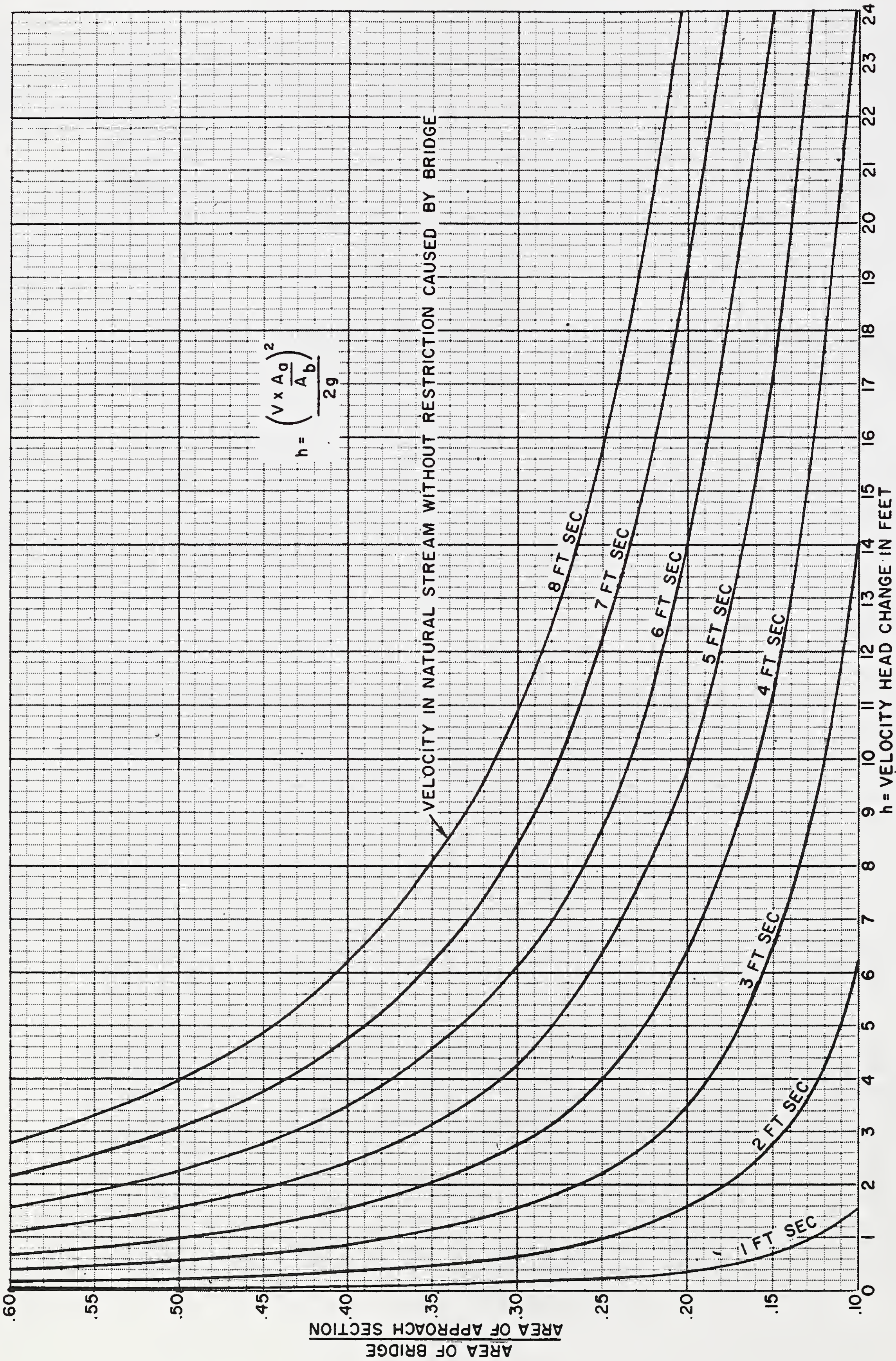


Exhibit 14-2. Estimate of head loss in bridges.

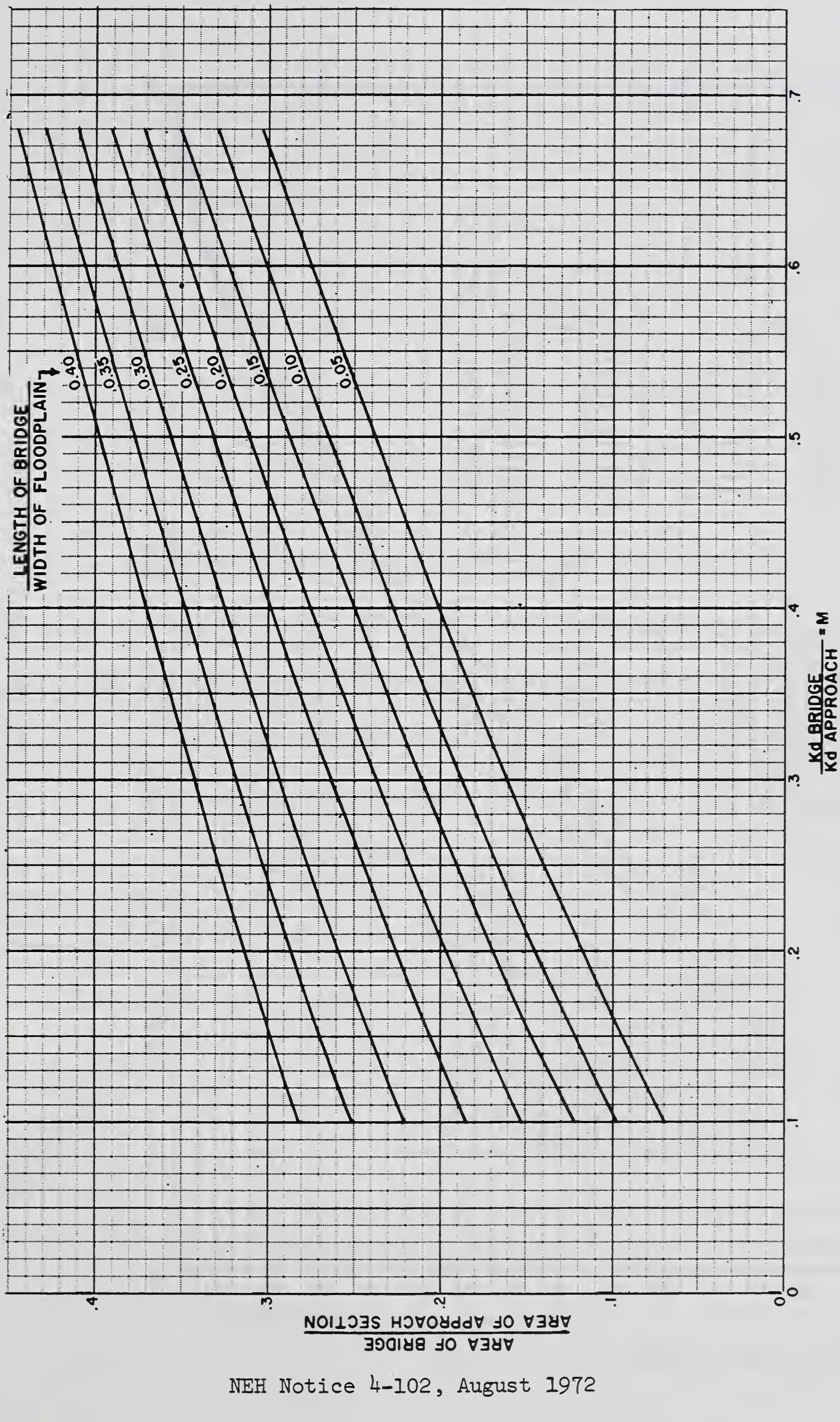


Exhibit 14-3. Estimate of M for use in BPR equation.

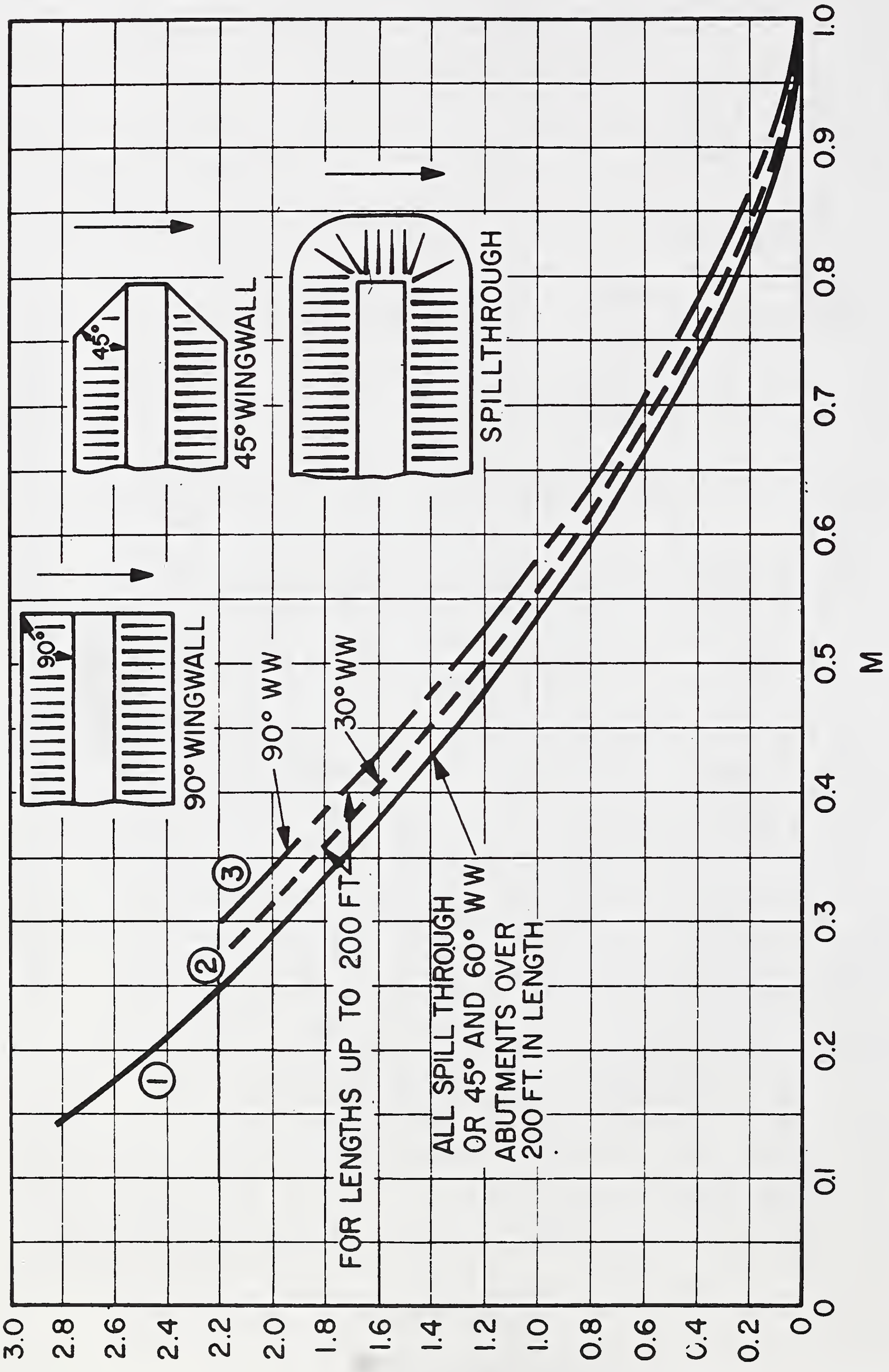


Exhibit 14-4. BPR base curve for bridges (K_b).

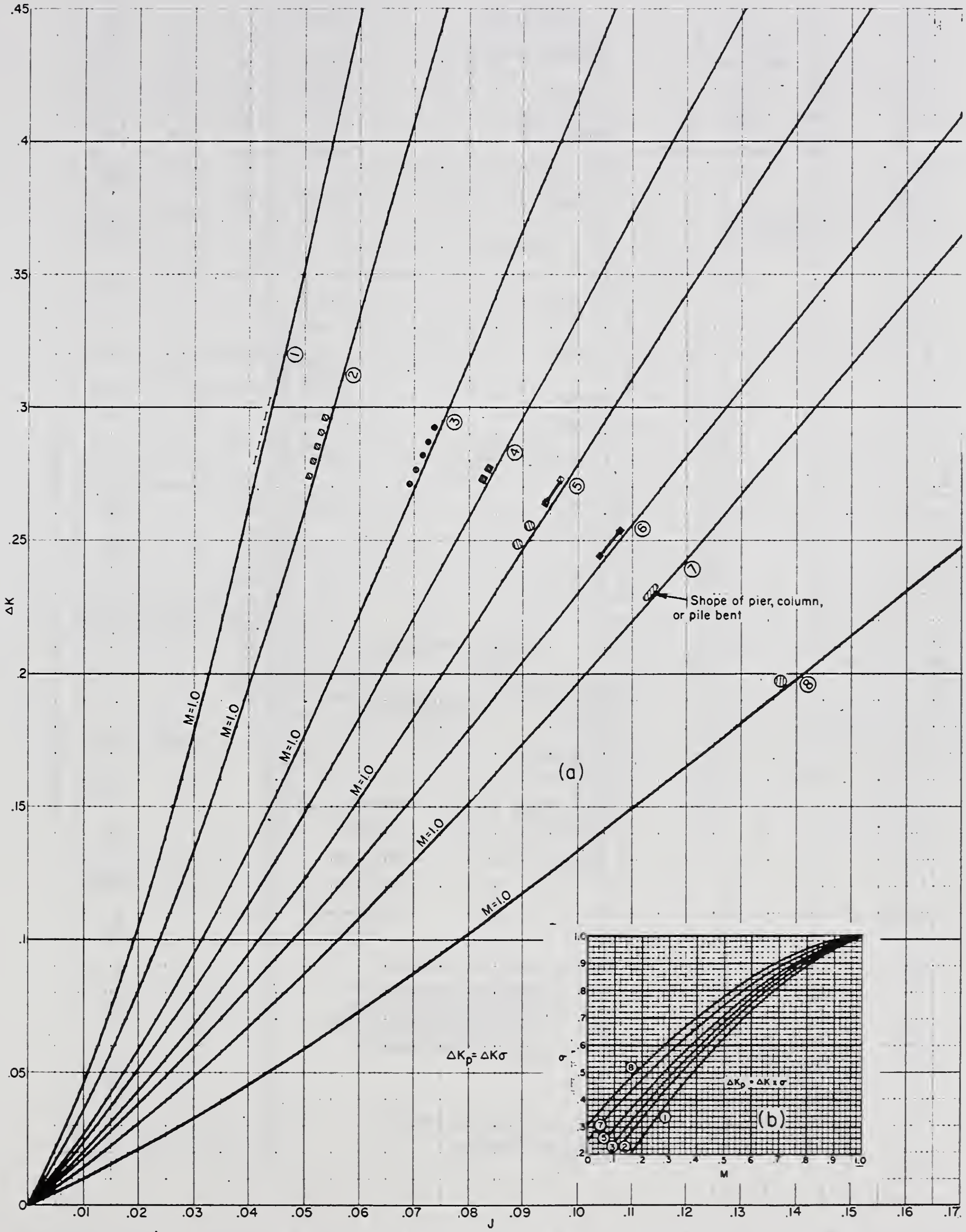


Exhibit 14-5. Incremental backwater coefficient for the more common types of columns, piers and pile bents.

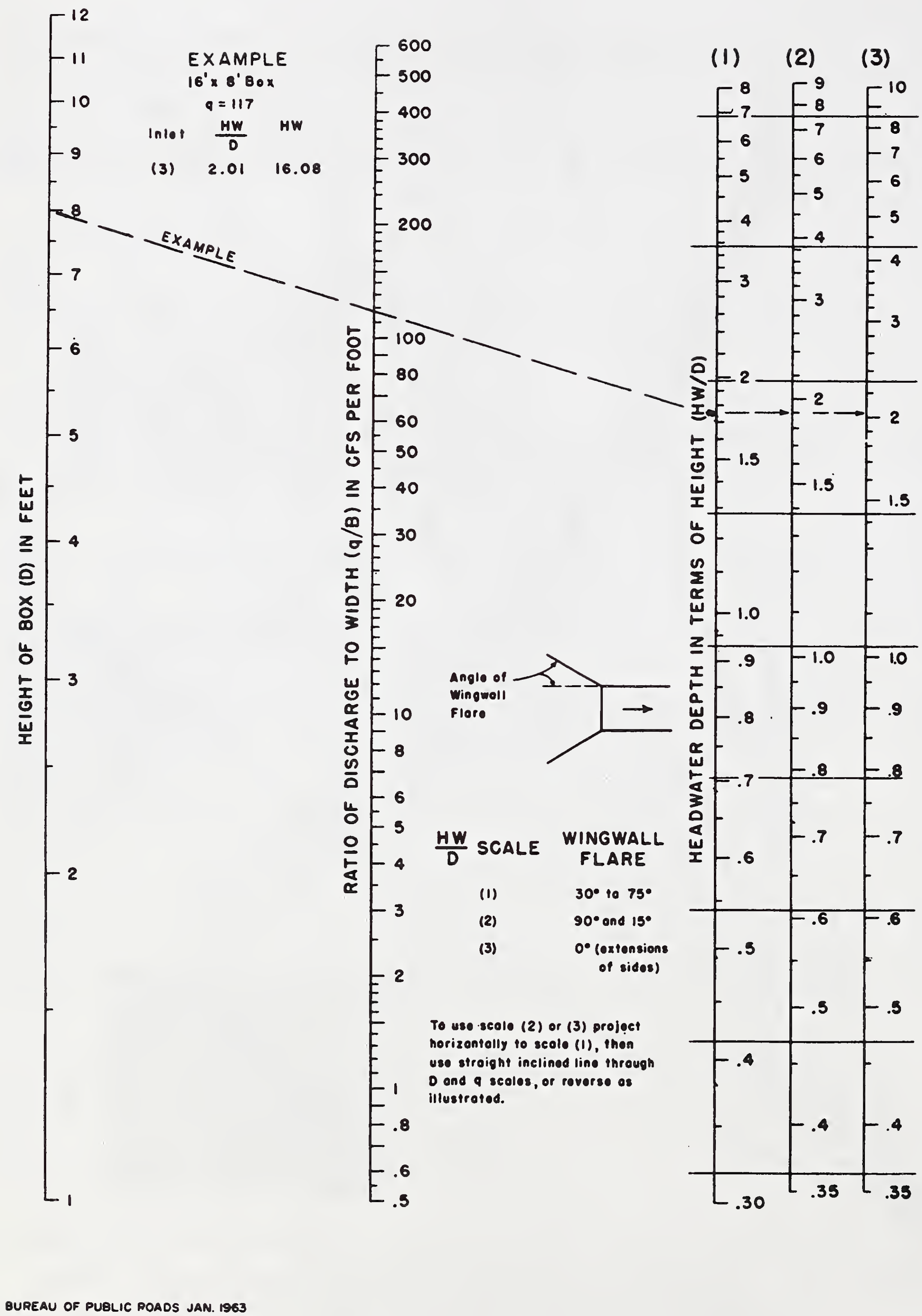


Exhibit 14-6. Headwater depth for box culverts with inlet control.

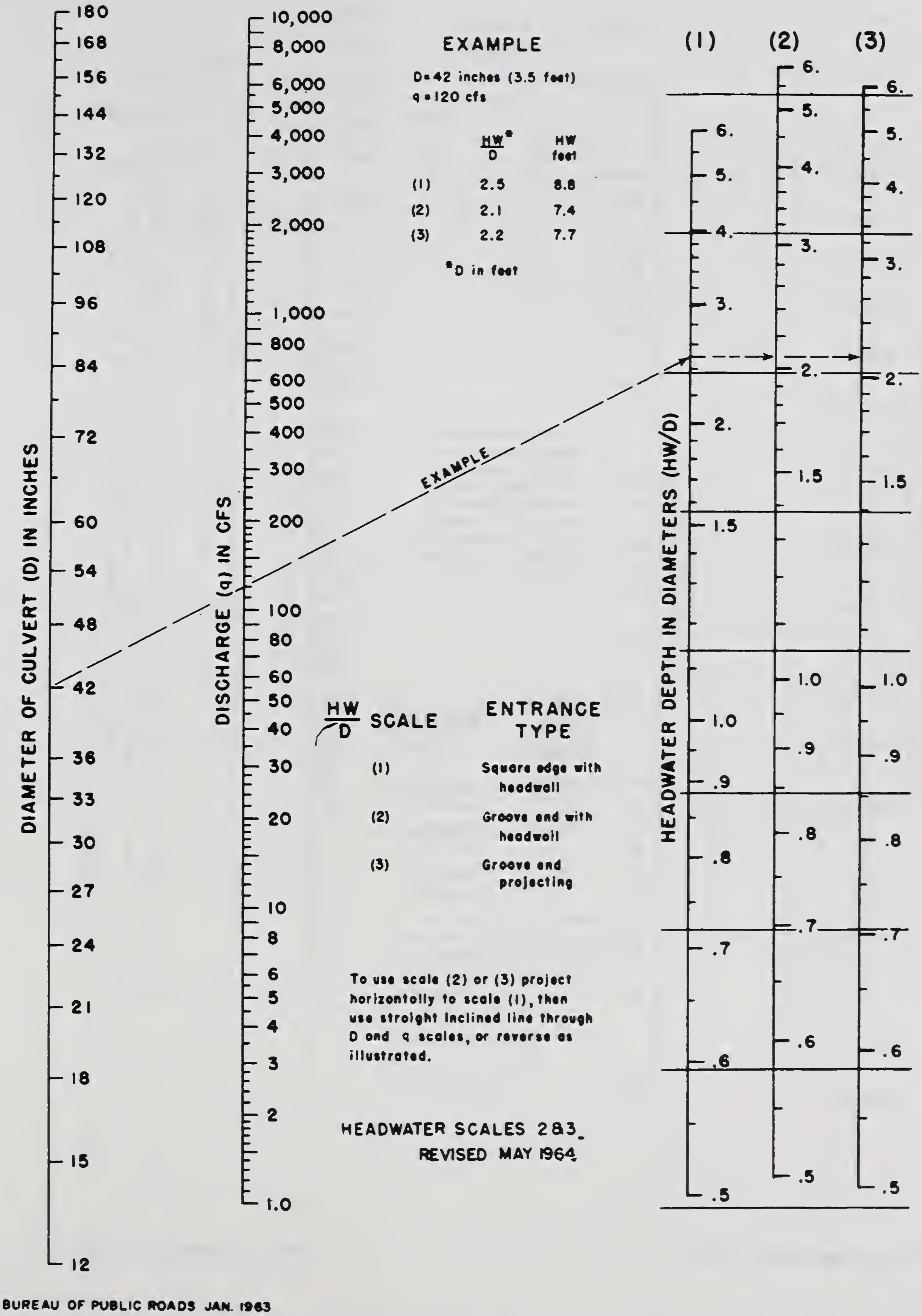


Exhibit 14-7. Headwater depth for concrete pipe culverts with inlet control.

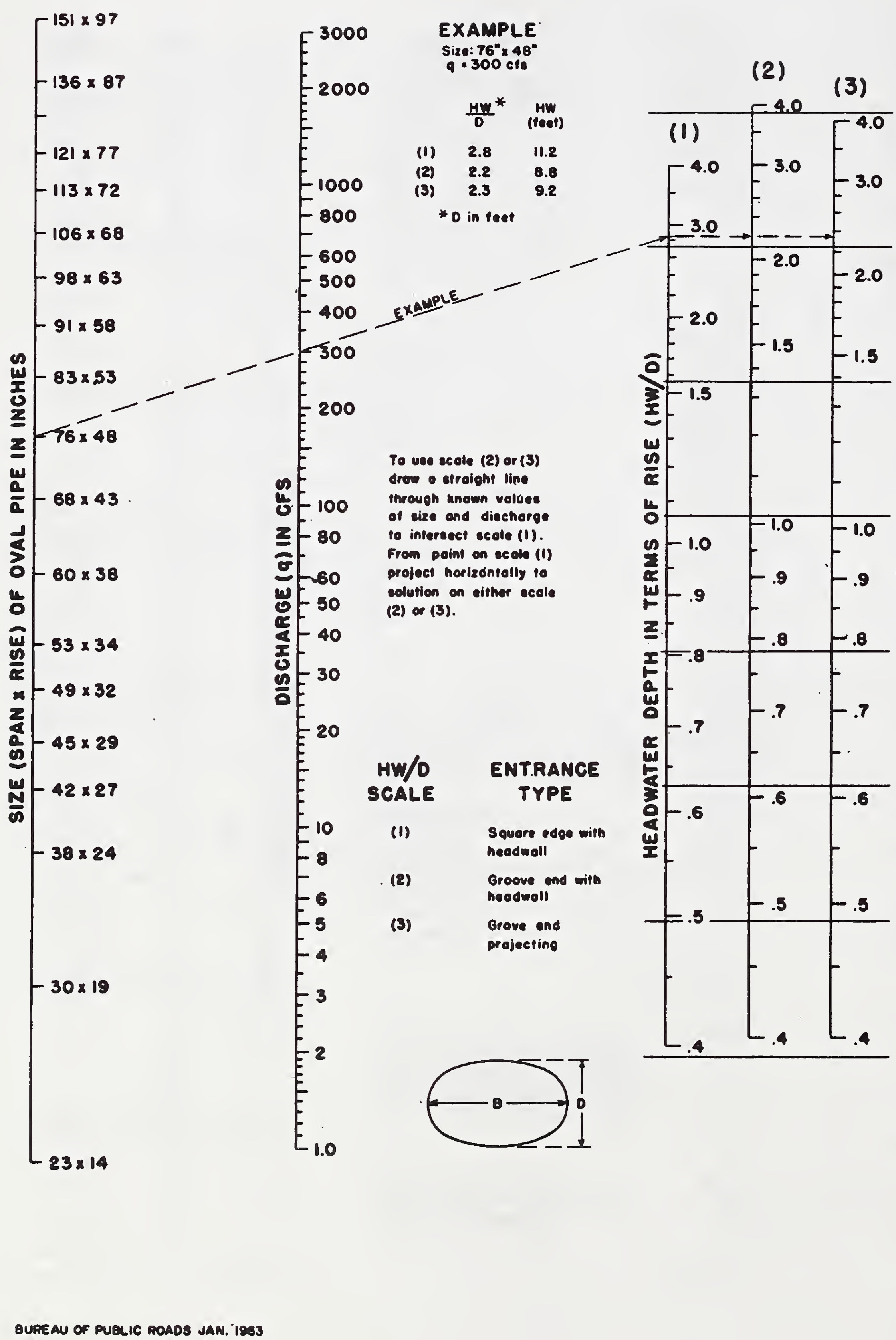


Exhibit 14-8. Headwater depth for oval concrete pipe culverts long axis horizontal with inlet control.

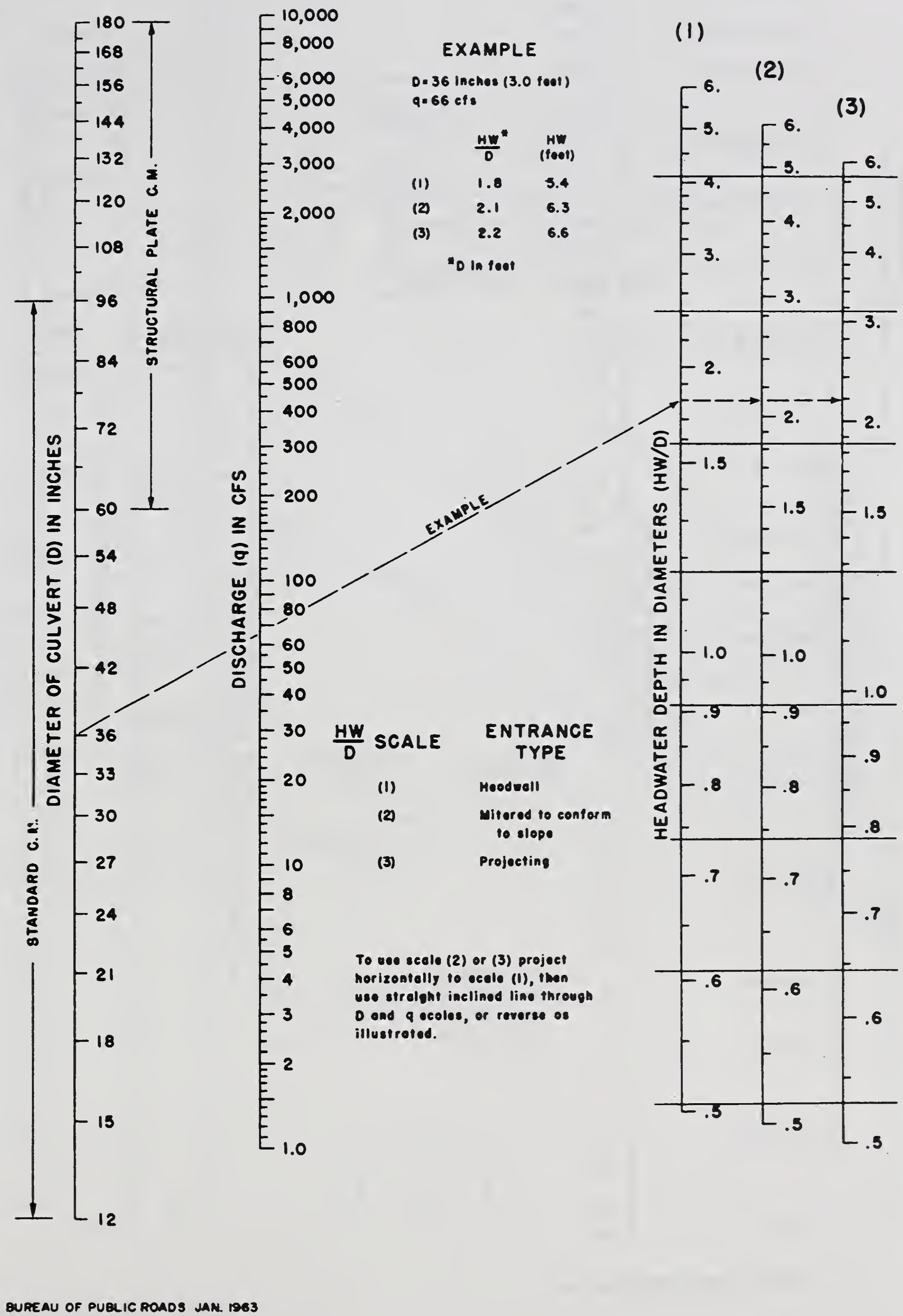


Exhibit 14-9. Headwater depth for C. M. pipe culverts with inlet control.

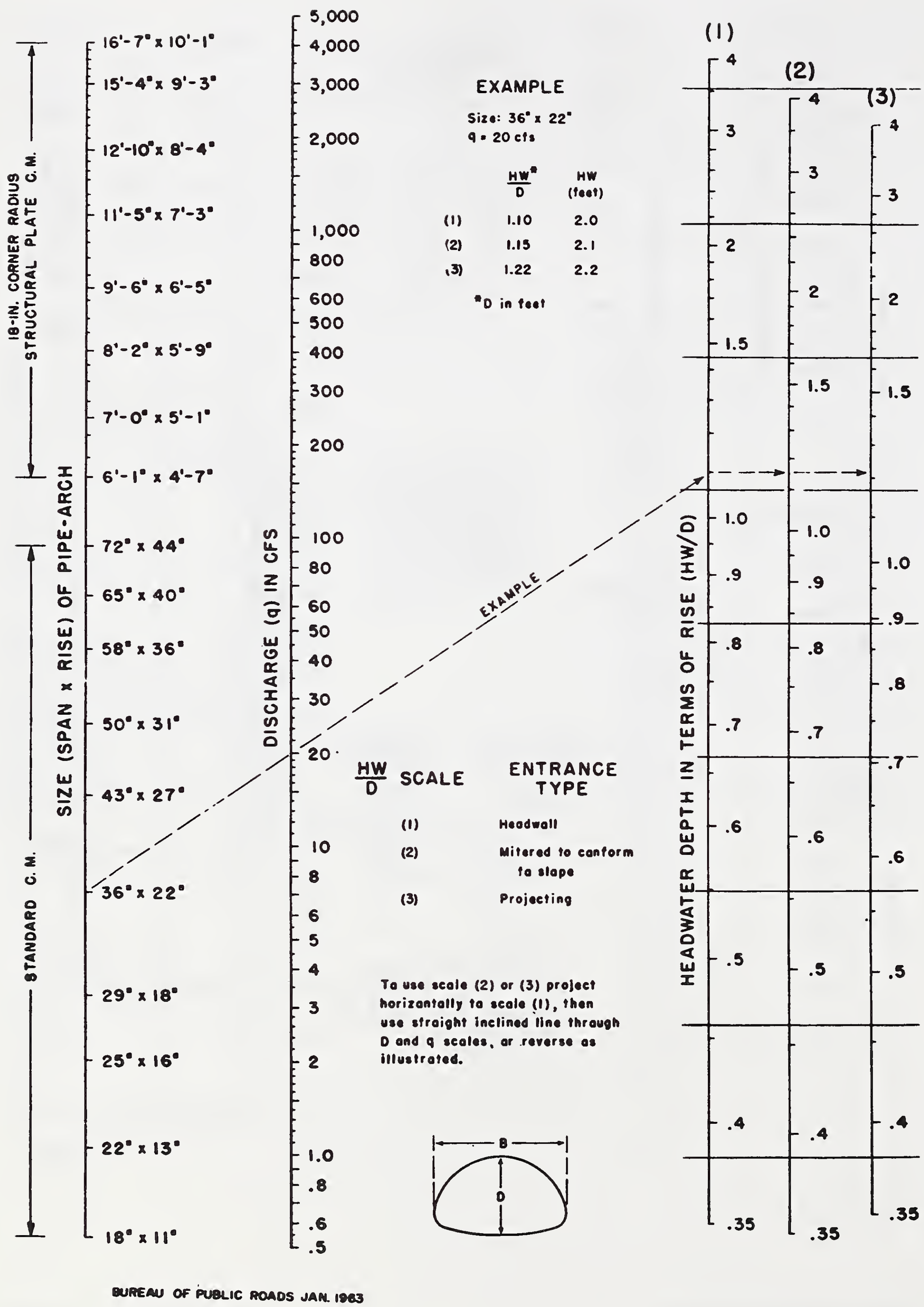


Exhibit 14-10. Headwater depth for C.M. pipe-arch culverts with inlet control.

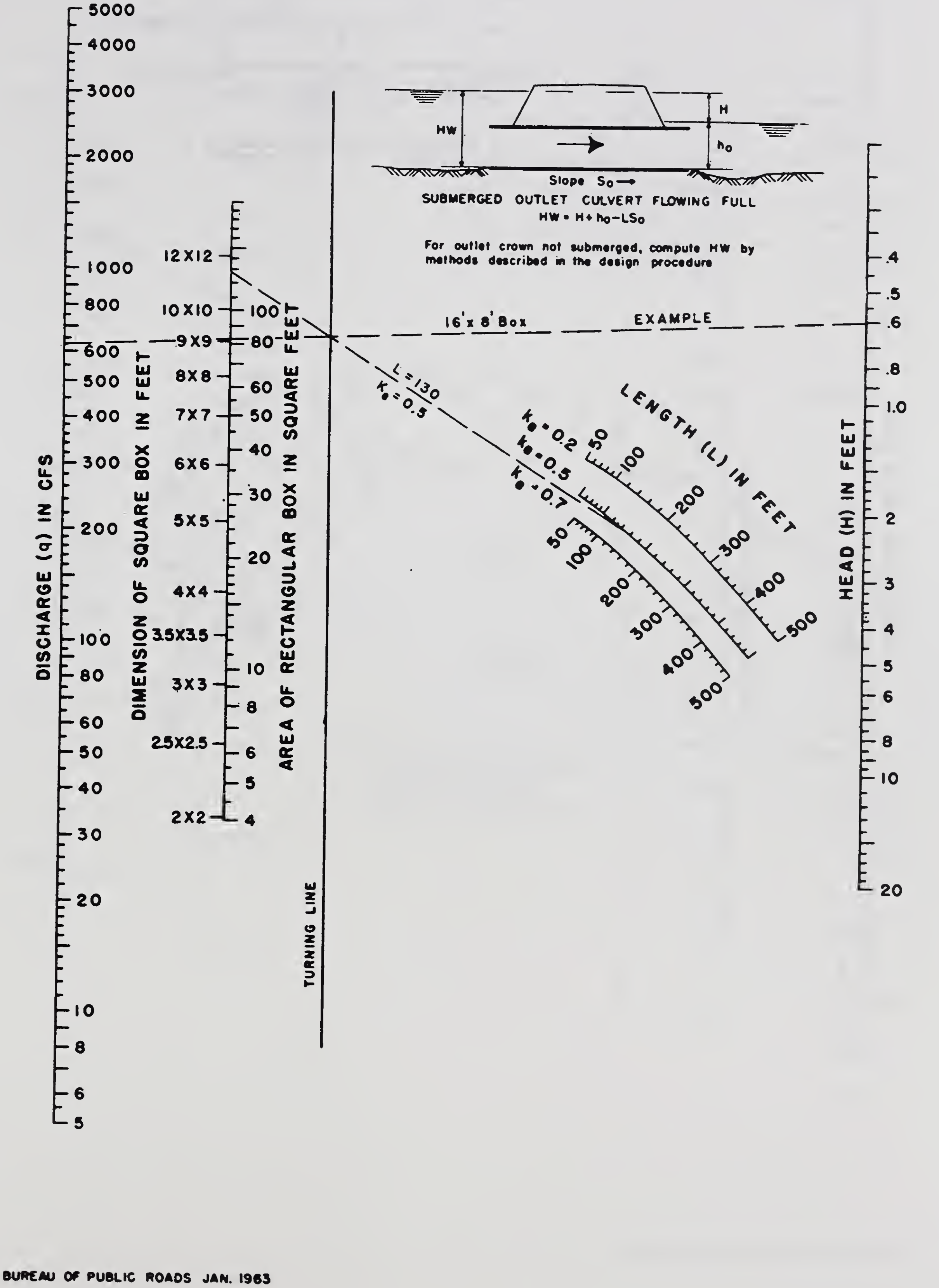


Exhibit 14-11. Head for concrete box culverts flowing full n = 0.012.

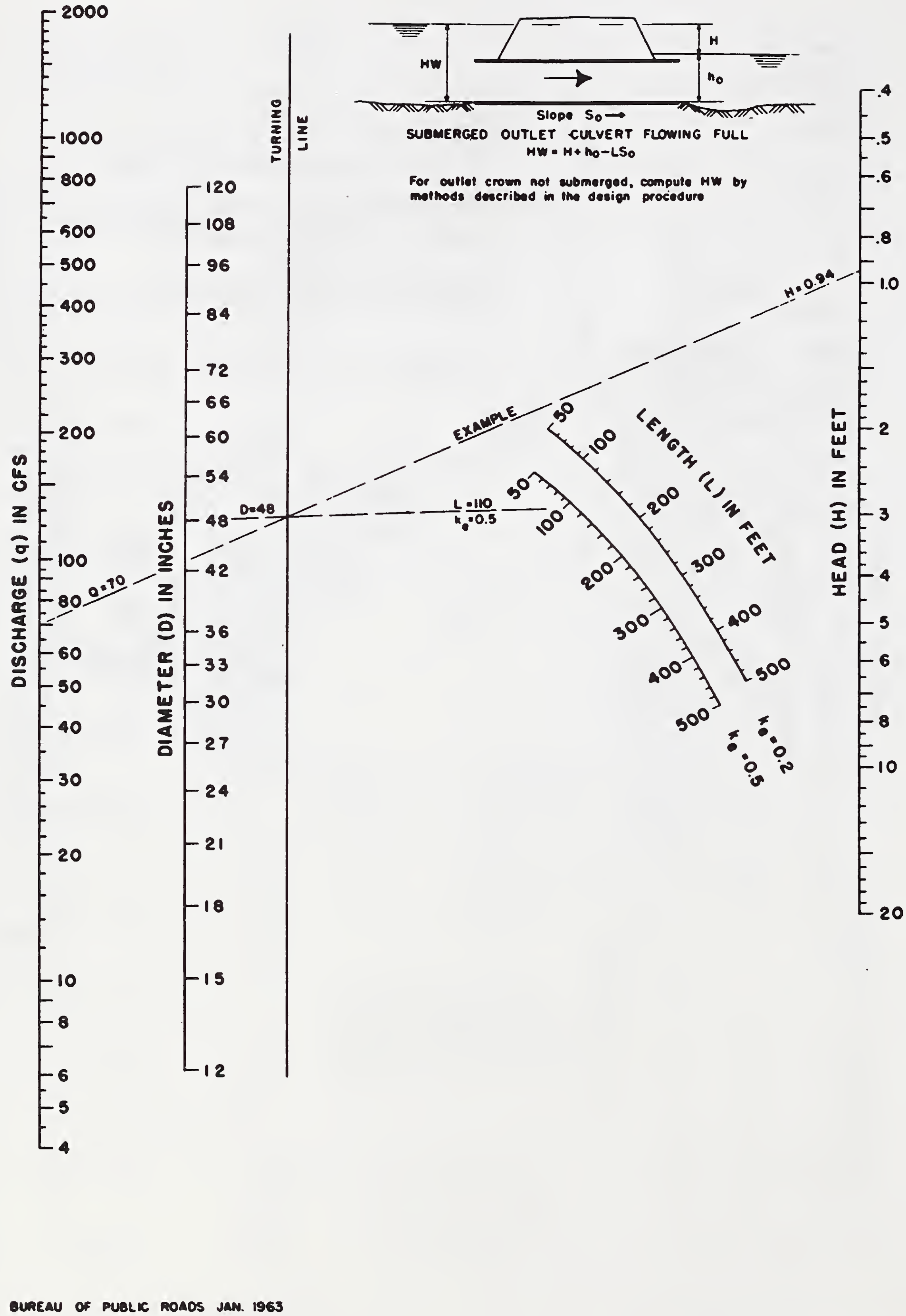


Exhibit 14-12. Head for concrete pipe culverts flowing full n = 0.012.

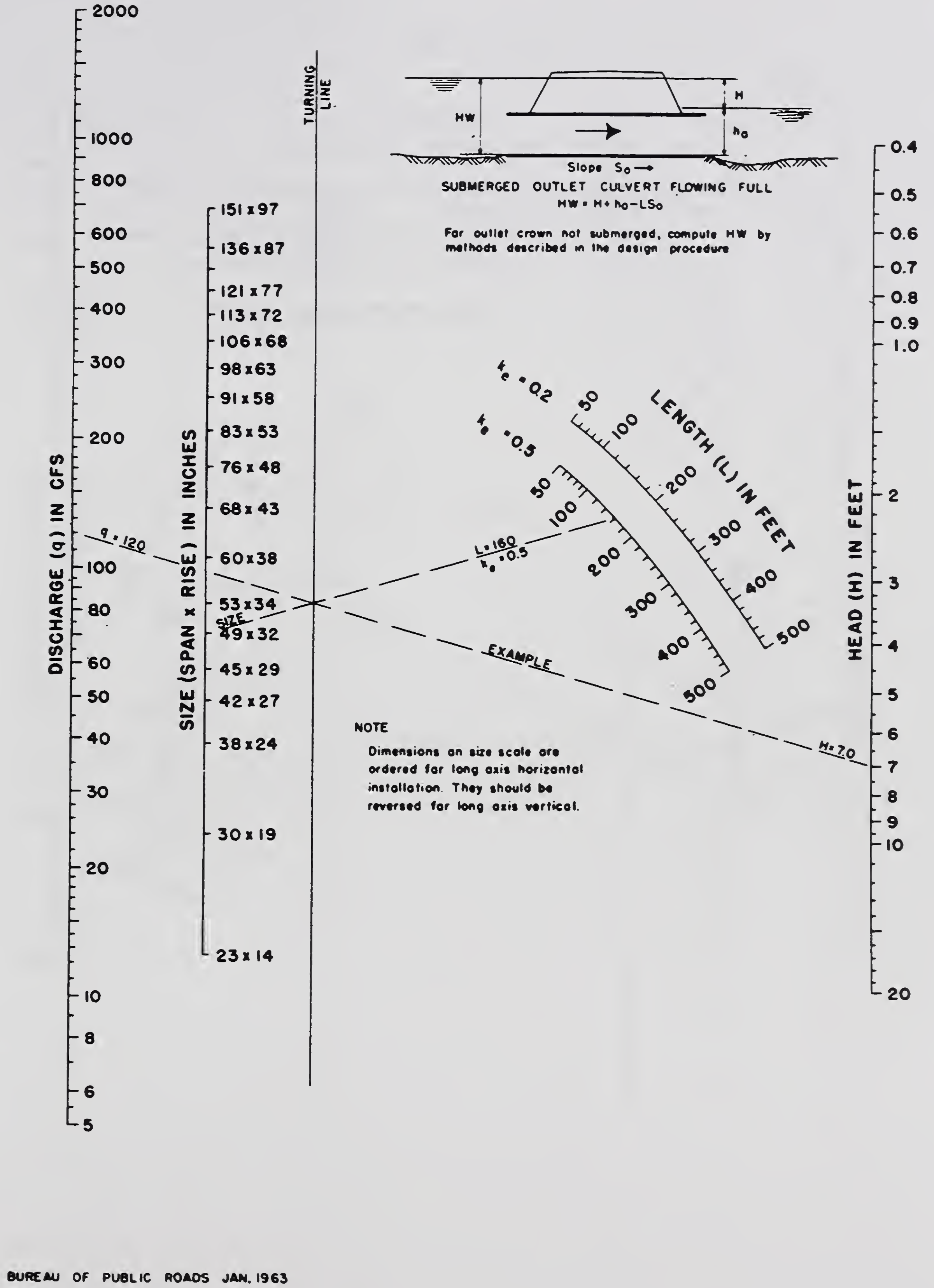


Exhibit 14-13. Head for oval concrete pipe culverts long axis horizontal or vertical flowing full n = 0.012.

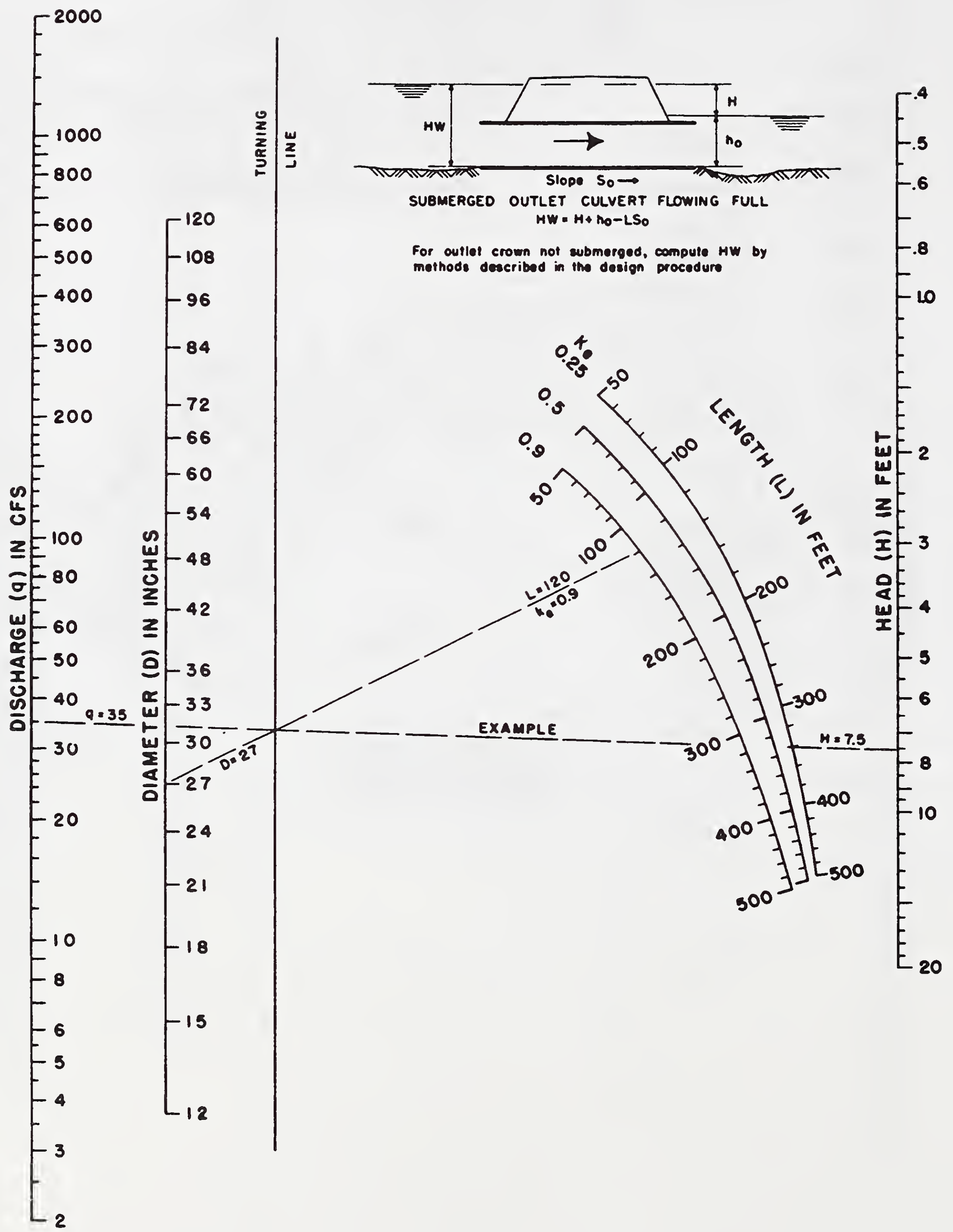


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Exhibit 14-14. Head for standard C. M. pipe culverts flowing full n = 0.024.

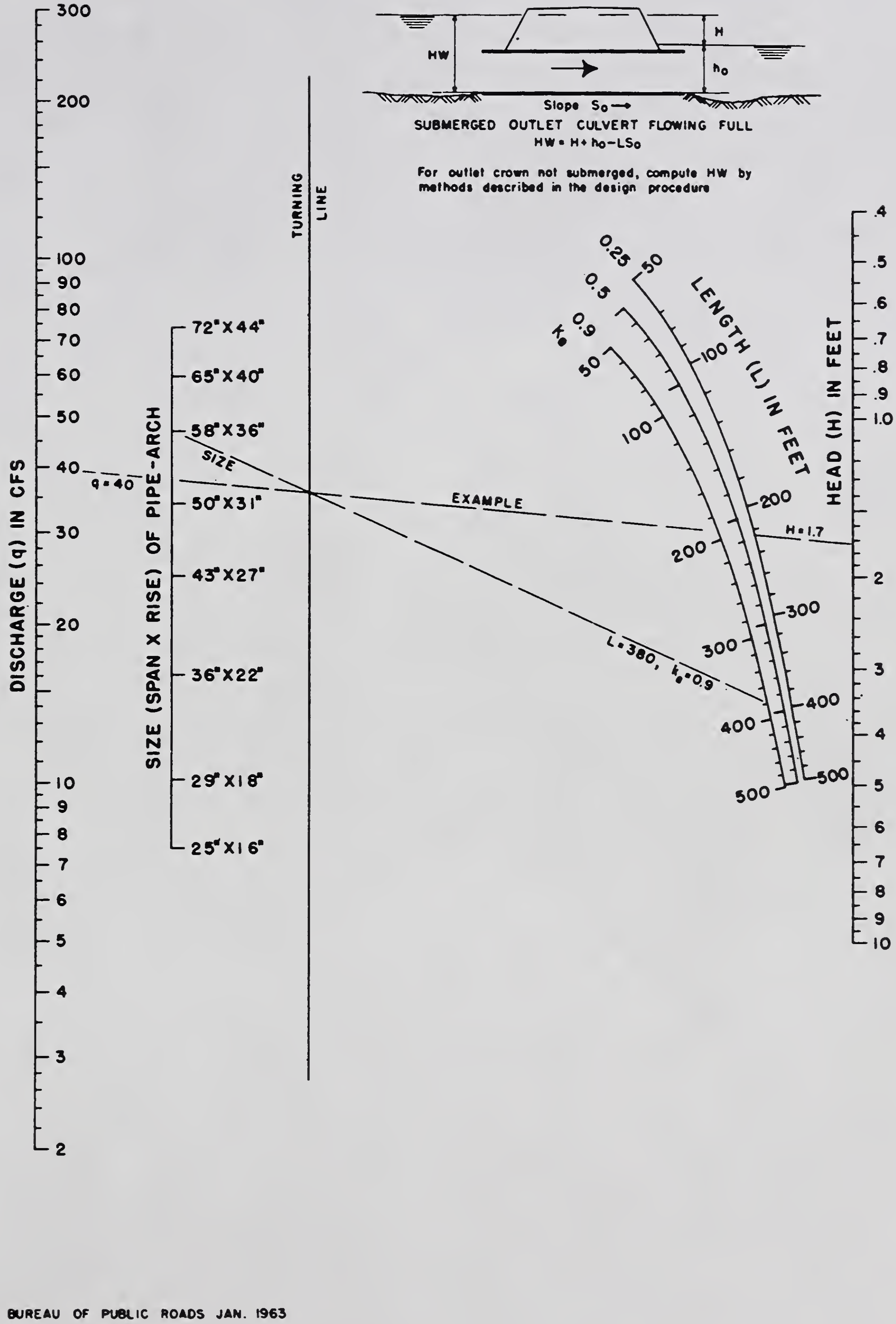
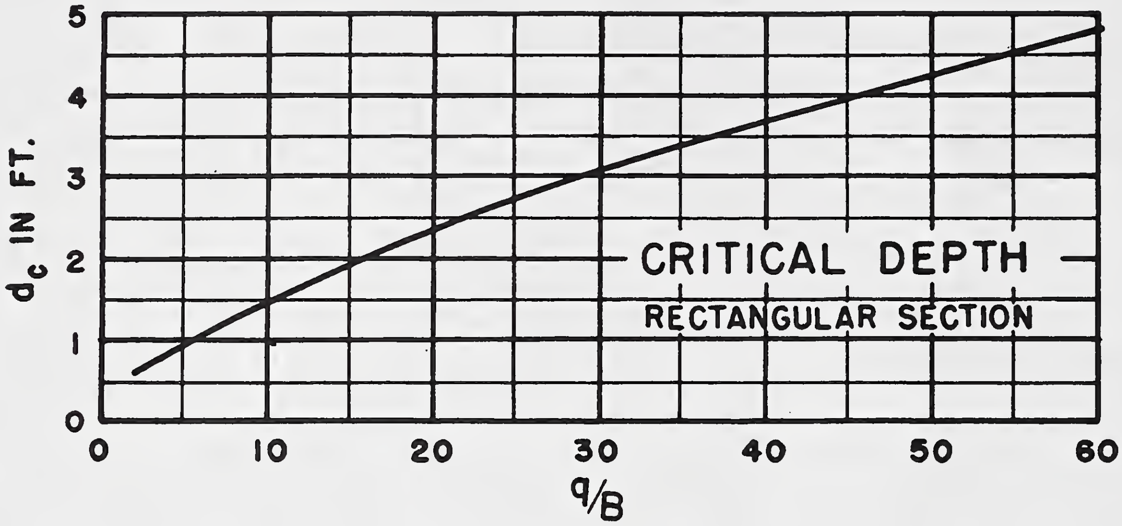


Exhibit 14-15. Head for standard C. M. pipe-arch culverts flowing full n = 0.024.



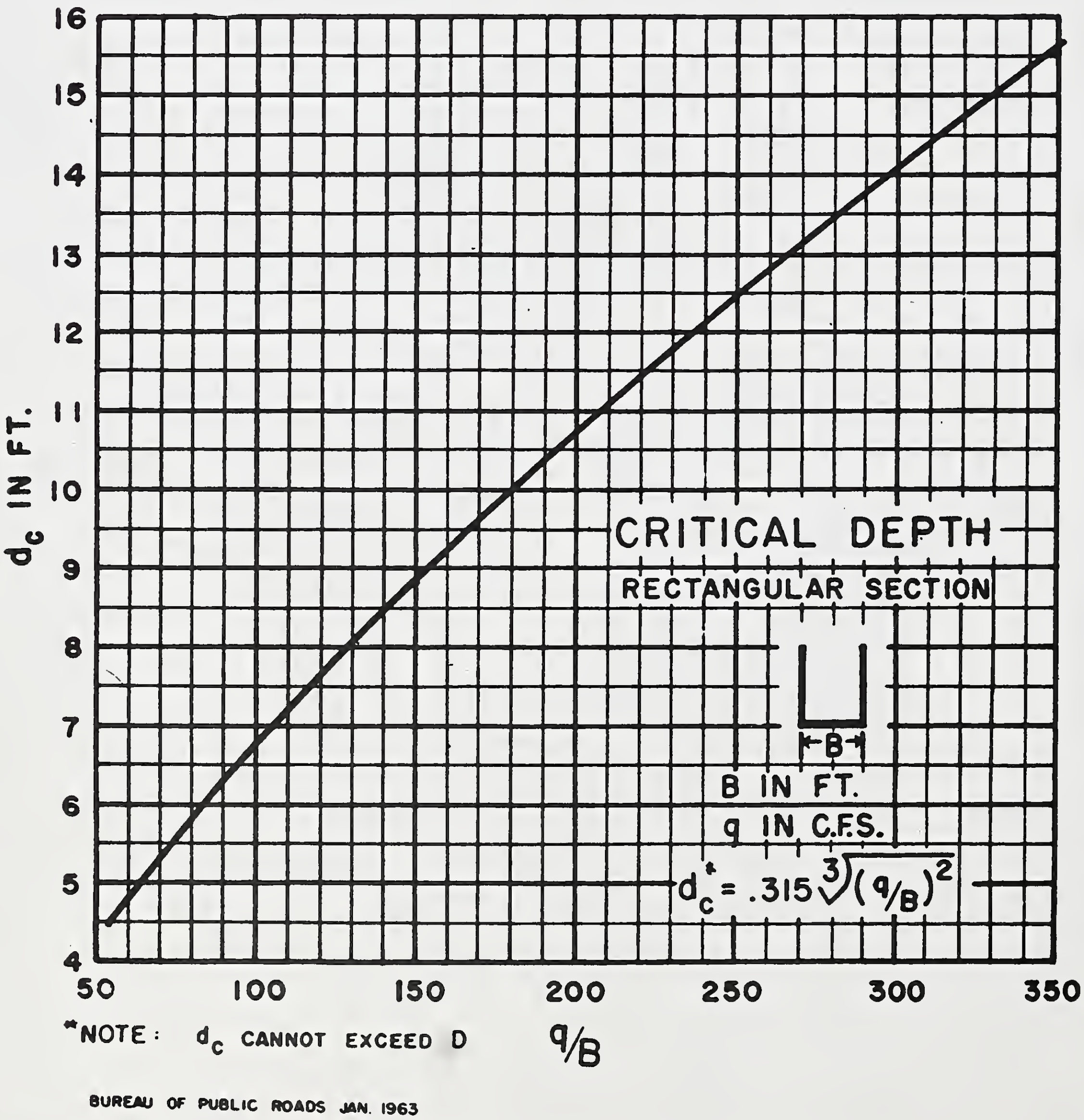


Exhibit 14-16. Critical depths-rectangular section.

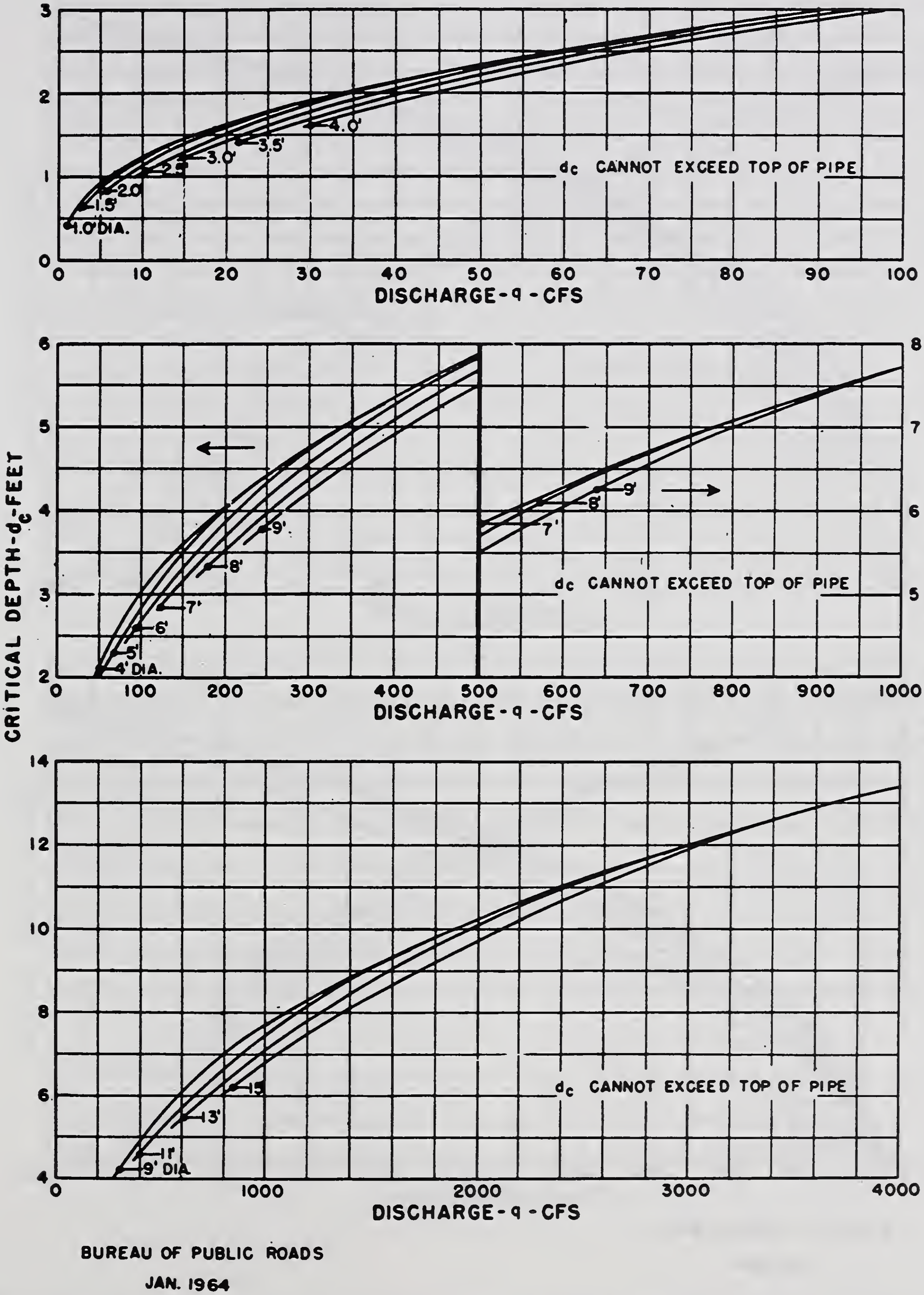


Exhibit 14-17. Critical depth. Circular pipe

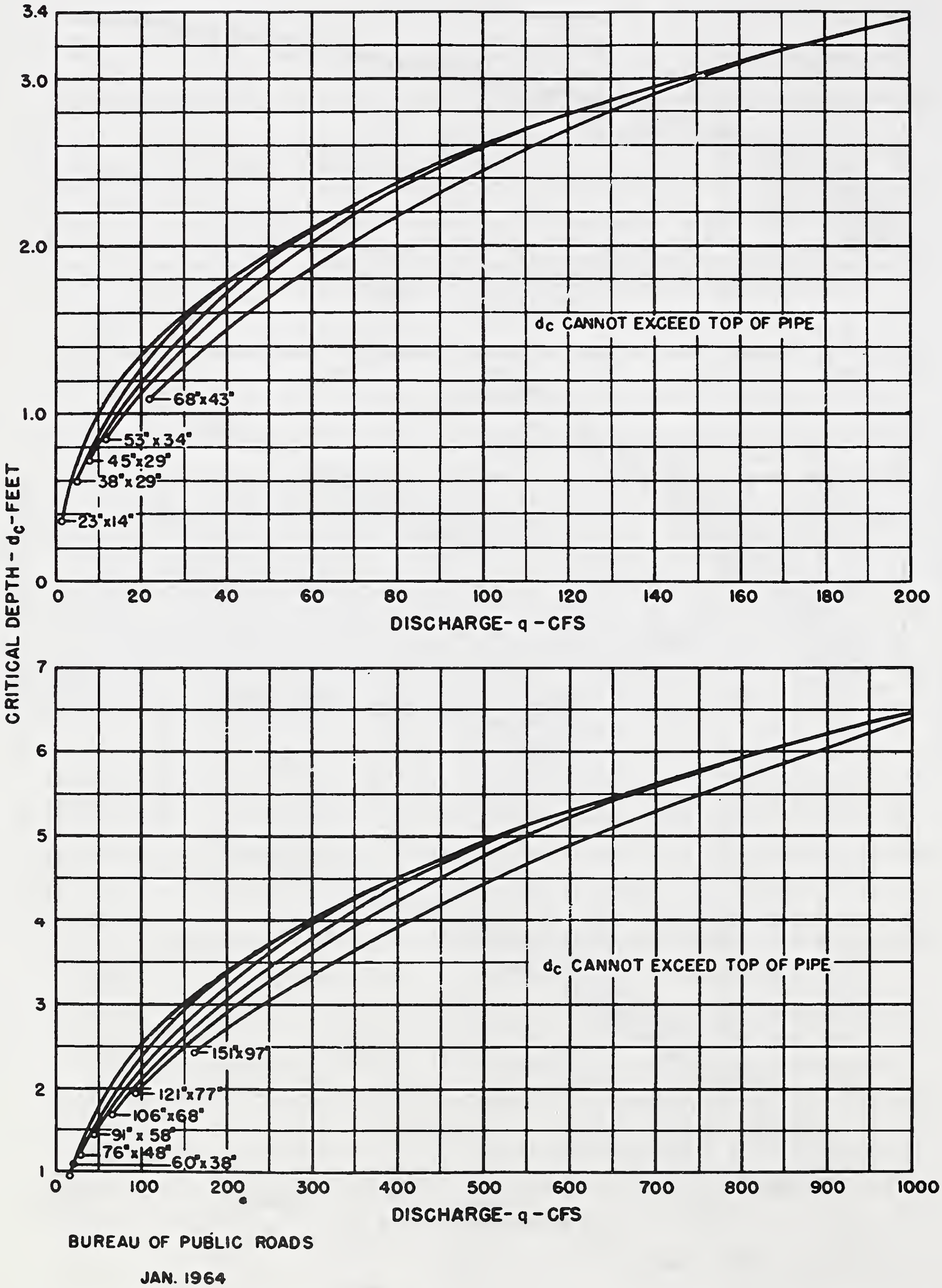
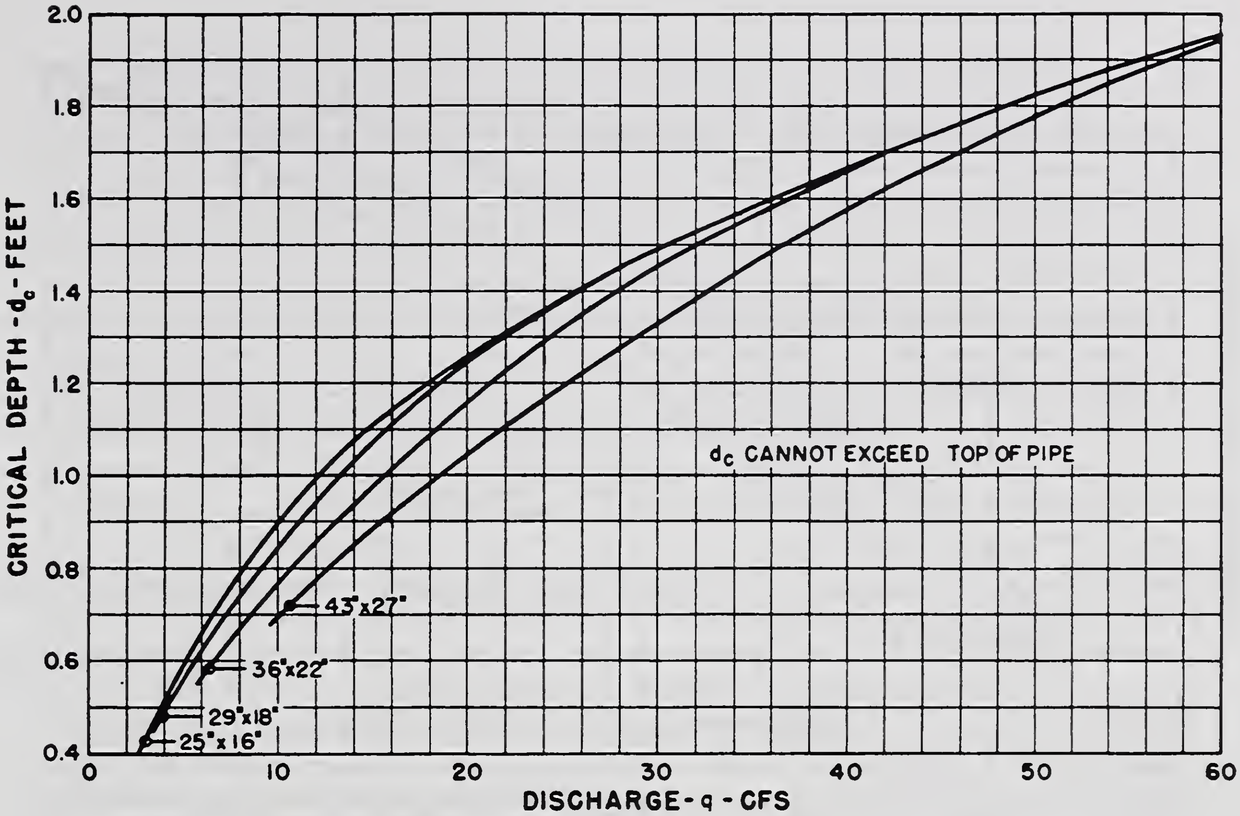


Exhibit 14-18. Critical depth. Oval concrete pipe. Long axis horizontal.



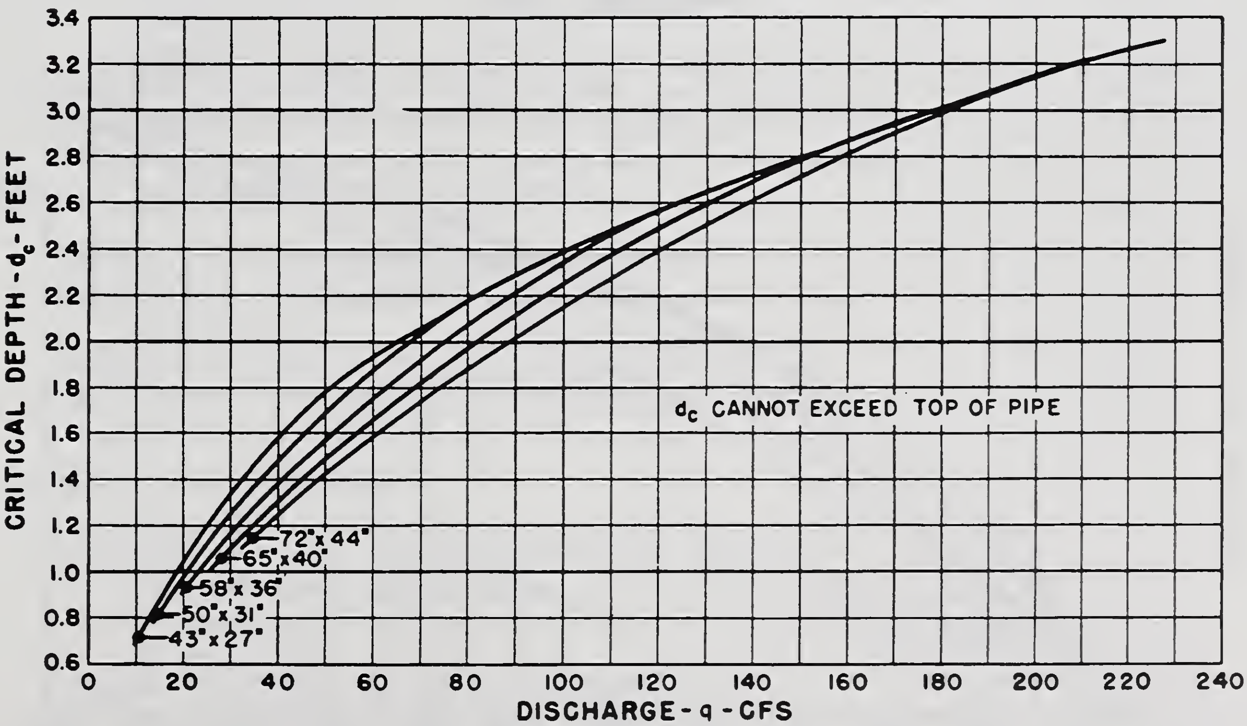


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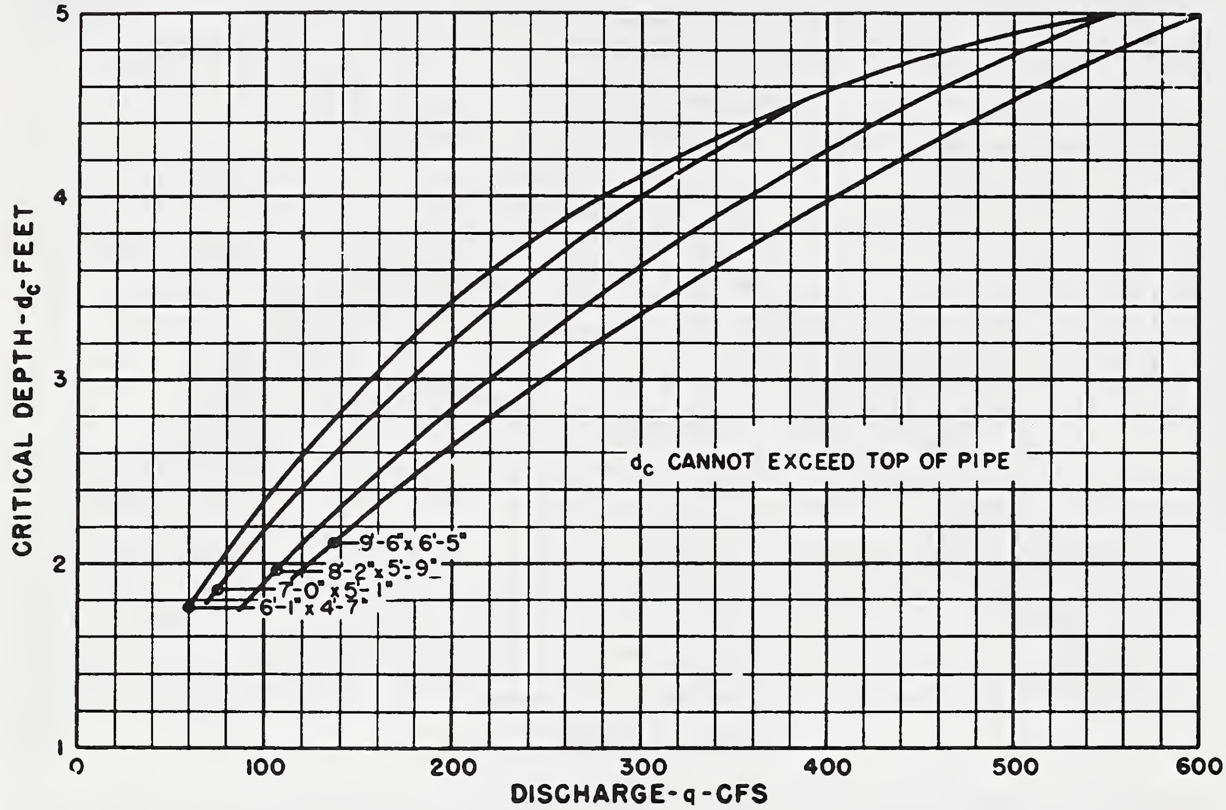
Exhibit 14-19. Critical depth. Standard C.M. pipe-arch.



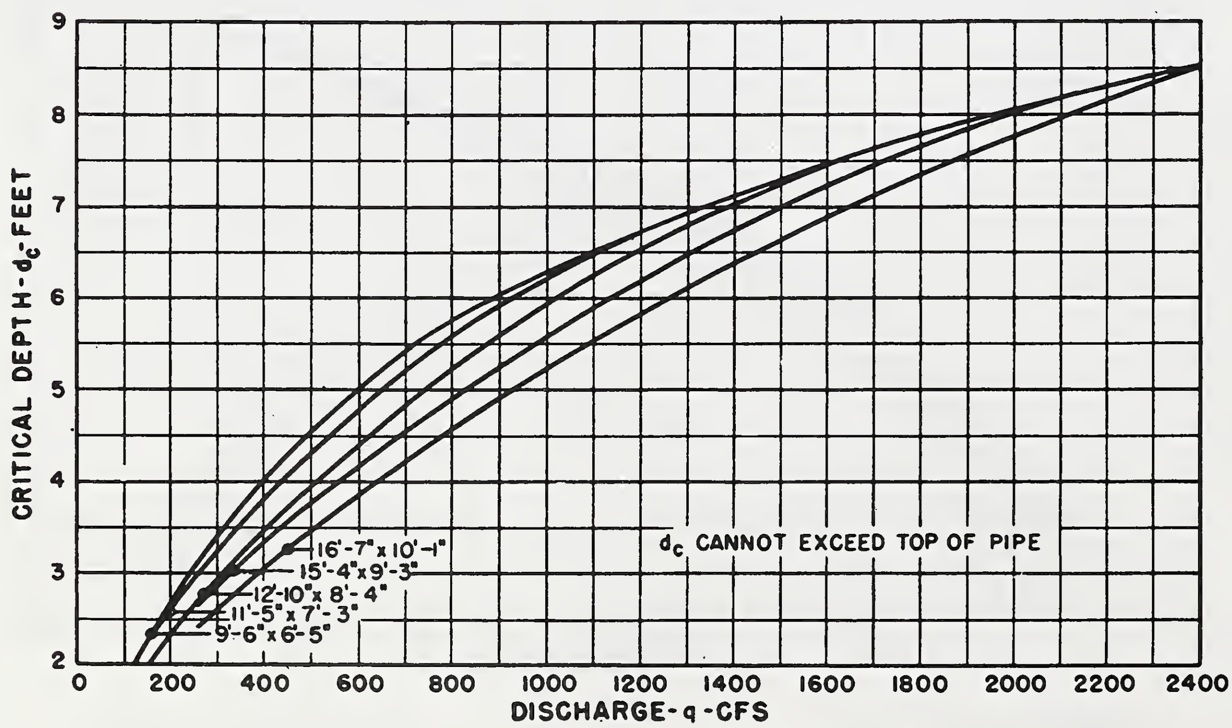


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Exhibit 14-20. Critical depth. Structural plate. C.M. pipe-arch.

Exhibit 14-21. Entrance loss coefficients.

Coefficient k_e to apply to velocity head $\frac{V^2}{2g}$ for determination of head loss at entrance to a structure, such as a culvert or conduit, operating full or partly full with control at the outlet.

$$\text{Entrance head loss } H_e = k_e \frac{V^2}{2g}$$

Type of Structure and Design of Entrance	Coefficient k_e
Pipe, Concrete	
Projecting from fill, socket end (groove-end)	0.2
Projecting from fill, sq. cut end	0.5
Headwall or headwall and wingwalls	
Socket end of pipe (groove-end)	0.2
Square-edge	0.5
Rounded (radius = 1/12D)	0.2
Mitered to conform to fill slope	0.7
End-Section conforming to fill slope	0.5
Pipe, or Pipe-Arch, Corrugated Metal	
Projecting from fill (no headwall)	0.9
Headwall or headwall and wingwalls	
Square-edge	0.5
Mitered to conform to fill slope	0.7
End-Section conforming to fill slope	0.5
Box, Reinforced Concrete	
Headwall parallel to embankment (no wingwalls)	
Square-edged on 3 edges	0.5
Rounded on 3 edges to radius of 1/12 barrel dimension	0.2
Wingwalls at 30° to 75° to barrel	
Square-edged at crown	0.4
Crown edge rounded to radius of 1/12 barrel dimension	0.2
Wingwalls at 10° to 25° to barrel	
Square-edged at crown	0.5
Wingwalls parallel (extension of sides)	
Square-edged at crown	0.7

NATIONAL ENGINEERING HANDBOOK

SECTION 4

HYDROLOGY

CHAPTER 15. TRAVEL TIME, TIME OF CONCENTRATION AND LAG

by

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NATIONAL ENGINEERING HANDBOOK

SECTION 4

HYDROLOGY

CHAPTER 15. TRAVEL TIME, TIME OF CONCENTRATION AND LAG

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HYDROLOGY

CHAPTER 15. TRAVEL TIME, TIME OF CONCENTRATION AND LAG

Introduction

There is a delay in time, after a brief heavy rain over a watershed, before the runoff reaches its maximum peak. This delay is a watershed characteristic called lag. It must be known before computing a peak flow time and rate for an ungaged watershed. Lag is related to time of concentration and may be estimated from it. Both lag and time of concentration are made up of travel times, which are also used in flood routings and hydrograph construction. This chapter contains methods for estimating travel time, lag, and time of concentration.

Types of Flow

Figure 15.1 shows four types of flow that may occur singly or in combination on a watershed.

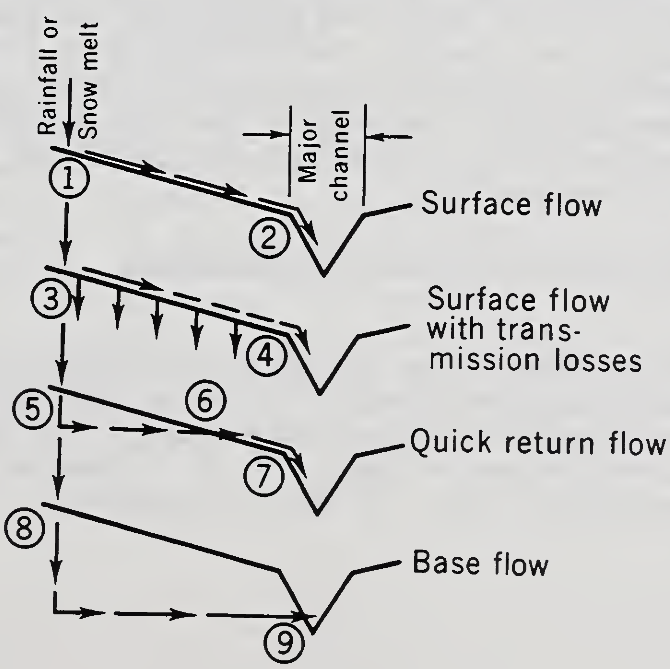

Figure 15. 1.--Types of flow

Surface Flow. - - Travel from point 1 to point 2 in figure 15.1 is along the surface of the watershed. This is surface runoff (also see Chapter 10). The flow takes place as overland flow or channel flow. This type is commonly discussed in hydrograph analysis but it seldom occurs in its ideal form.

Surface Flow with Transmission Losses. - - Water traveling toward the watershed outlet is infiltrated into the soil or channel material. This type is common in arid, semiarid, and subhumid climates. When the infiltration takes place in a channel, it is called a transmission loss (see Chapter 19). The distance from point 3 to point 4 in figure 15.1 will depend on the amount of runoff, the moisture characteristics of the soil and on hydraulic features of the flow.

Interflow or Quick Return Flow. - - Water infiltrated at point 5, figure 15.1, eventually returns to the surface at point 6, continuing as surface flow to point 7. This flow reappears rapidly in comparison to base flow and is generally much in excess of normal base flow. Springs or seeps that add to flood flows are of this type. It is common in humid climates and in watersheds with soils of high infiltration capacities and moderate to steep slopes.

Base Flow. - - Rainfall entering at point 8, figure 15.1, goes directly to the ground water table, eventually entering a stream at point 9. This type of flow has little effect on flood peaks in small watersheds. However, if it is a factor, it is usually added to the hydrograph as a constant discharge.

Measurement of flow

On figure 15.1, flows from points 1 to 2, 3 to 4, and 6 to 7 can be measured directly (see Chapter 14). Flows from points 5 to 6 and 8 to 9 are usually determined indirectly by storm and hydrograph analyses or by field observation of rainfall and runoff. The distance from point 3 to 4 in figure 15.1 will depend on the amount and rate of runoff, moisture condition in the soil and the hydraulic features of the flow. Such water cannot be measured except indirectly by analyses of precipitation, soil moisture movements, and evapotranspiration.

Travel Time, Lag and Time of Concentration

Travel time

Travel time (T_t) is the time it takes water to travel from one location in a watershed to another location downstream. The travel may occur on the surface of the ground or below it or in a combination of the two. T_t is affected by hydraulic factors and storage. It is a component part of lag (L) and time of concentration (T_c). It can be estimated by equation 15.1.

$$T_t = \frac{\ell}{3600\ V} \quad \text{. . . . . Eq. 15.1}$$

Where T_t = travel time in hours
ℓ = hydraulic length in feet
V = velocity in feet per second

Lag

The lag (L) of a watershed may be thought of as a weighted time of concentration. If for a given storm the watershed is divided into increments, and the travel times from the centers of the increments to the main watershed outlet are determined, then the lag is:

$$L = \frac{\Sigma(a_x\ Q_x\ \ T_{t_x})}{A\ \ Q_a} \quad \text{. Eq. 15.2a}$$

$$L = \frac{\Sigma(a_x\ Q_x\ T_{t_x})}{\Sigma(a_x\ Q_x)} \quad \text{. Eq. 15.2b}$$

where L = lag in hours

a_x = the x-th increment of watershed area in square miles

Q_x = runoff in inches from area a_x

T_{t_x} = travel time in hours from the center of a_x to the point of reference

A = total area of the watershed above the point of reference

Q_a = average runoff in inches from the total area (A), or $\Sigma(a_x\ Q_x)/A$

Equation 15.2 will give the watershed lag for all the types of flow shown in figure 15.1. However, the difficulties of obtaining accurate estimates of underground flow rates and paths limits the use of the equation. Instead, the approach in general practice is to develop a hydrograph for each of the subareas (A_x) in equation 15.2 and route the hydrographs downstream to the point of reference. The subareas are usually a subdivision of a hydrologic unit as described in Chapter 6. A lag time (L) or time of concentration (T_c) is usually estimated for each hydrologic unit by one of the methods in this Chapter. Hydrographs are then developed for each by a method of Chapter 16 and routed to the point of reference by a method of Chapter 17.

In simple hydrograph analysis, lag is the time from the center of mass of excessive rainfall to the peak rate of runoff (see Chapter 16). When combinations of flow occur together, a compound hydrograph with more than one peak and lag time may result. Ideally the various types of flow should be separated for lag analysis and combined at the end of the study. Water exists in a watershed system as a shapeless mass occurring in varying combinations of surface runoff, interflow and ground water flow. These components are characterized by the path the water takes from where it is generated to the point of reference, downstream. The velocity distribution varies both horizontally and vertically and lacks constant boundaries, thus the flow pattern cannot be evaluated by simple hydraulic analysis. In practice, lag is usually determined only for the direct runoff portion of flow.

The role of channel and valley storage are important in the development and translation of a flood wave and the estimation of lag. Both the hydraulics and storage may change from storm to storm, so that an average lag may have a large error. The problem of evaluating lag is sufficiently complex that theoretical hydraulic analysis must be complemented with a hydrologic appraisal of the relative effect of basin characteristics in order to make the best estimate.

Time of concentration

This is the time it takes for runoff to travel from the hydraulically most distant part of the storm area to the watershed outlet or other point of reference downstream. In hydrograph analysis, T_c is the time from the end of excessive rainfall to the point on the falling limb of the hydrograph (point of inflection) where the recession curve begins (see Chapter 16). T_c is generally understood as applying to surface runoff.

The implication in the definitions of L and T_c, that the time factor is only a case of calculating a theoretical velocity of a segment of water moving through a hydraulic system, is an over-simplification. As with lag, T_c may vary because of changes in hydraulic and storage conditions.

Estimating T_c, T_t and L

Each method presented here is in effect a short-cut from which one or more watershed characteristics have been omitted. It is a good practice to consider more than one method, choosing the one that best fits the characteristics of a given watershed. It is not worthwhile averaging estimates made using two or three methods. Instead, the method that appears most applicable because of field and data conditions should be used.

Field observations
At the time field surveys to obtain channel data are made, there is a need to observe the channel system and note items that may affect channel efficiency. Observations such as the type of soil materials in the banks and bottoms of the channel; an estimate of Manning's roughness coefficient; the apparent stability or lack of stability of channel; indications of debris flows as evidenced by deposition of coarse sediments adjacent to channels, size of deposited materials, etc., may be significant.

Indications of channel stability can sometimes be used to bracket the range of velocities that normally occur in the stream channels. Because high sediment concentrations frequently affect both channel velocities and peak rates of runoff, it is important to note when this potential exists.

Intensity of investigations
The purpose for which a study is made is a guide to the amount of work that should be done in securing data to serve as a basis for estimating T_c (Chapter 6). Where the hydrograph is to be the basis for design or for an important conclusion in planning, sufficient surveys should be made to serve as a basis for (a) dividing the main drainage course into reaches that are approximately uniform as to channel sizes, slopes and characteristics and (b) determining average cross sections, roughness coefficients and slopes for each reach. Where the hydrograph is to be the basis for preliminary conclusions, T_c may be estimated by taking the travel distance from maps or aerial photographs and estimating average velocity from general knowledge of the approximate sizes and characteristics of channels in the area under consideration.

Many natural streams have considerable sinuosity, meander, etc. as well as overfalls and eddies. Tendencies are therefore, to underestimate the length of channels and overestimate average velocities through reaches.

Stream hydraulics for estimating travel time and T_c
This method is recommended for the usual case where no usable hydrographs are available. This procedure is most applicable for areas where surface runoff predominates. It can result in too short of T_c for areas where interflow and ground water flow are a major part of runoff.

Stream or valley lengths and flow velocities are used, being taken from field survey data. It is assumed the stream has been divided into reaches.

1. Estimate the 2-year frequency discharge in the stream. When this cannot be done, use the approximate bankfull discharge of the low flow channel.

2. Compute the average velocity. In watersheds with narrow flood plains where the depth of overbank flow may be 10 to 20 feet during a major flood event, it may be desirable to use correspondingly higher velocities for frequencies of 10 to 100 years or greater.

3. Use the average velocity and the <u>valley length</u> of the reach to compute the travel time through the reach by equation 15.1.

4. Add the travel times of step 3 to get the T_c for the watershed. Use of the low flow channel bankfull discharges with valley lengths is a compromise that gives a T_c for average floods. For special cases (channel design, for instance) use whatever average velocities and lengths are appropriate.

In most cases the hydraulic data do not extend upstream to the watershed ridge. The remaining time (to add in step 4) can be estimated by adding the time obtained by the <u>upland method</u> or the T_c obtained by the <u>curve number method</u>. See figures 15.2 and 15.3 respectively. Use the one most applicable to the upper watershed characteristics.

Lag may be estimated in terms of T_c using the empirical relation:

$$L = 0.6\ T_c \quad \text{. . . . .} \quad \text{Eq. 15.3}$$

This is for average natural watershed conditions and for an approximately uniform distribution of runoff on the watershed. When runoff is not uniformly distributed the watershed can be subdivided into areas within which the runoff is nearly uniform, enough so that equation 15.3 can be applied.

<u>Upland method</u>
Types of flow considered in the upland method are: overland; through grassed waterways; over paved areas; and through small upland gullies. Upland flow employed in this method can be a combination of these various surface runoff conditions. The velocity is determined using figure 15.2.

The most remote segment of runoff that becomes part of the total time of concentration may occur in wide sheets overland rather than in defined channels. This type of flow is of practical importance only in very small watersheds because runoff is usually concentrated into small gullies or terrace channels within less than a thousand feet of its origin. The velocity of overland flow varies greatly with the surface cover and tillage as demonstrated in figure 15.2.

Surface runoff along terrace channels is another type of upland flow. The velocity and distance of flow that relate to time of concentration is based on the terrace gradient and length. A velocity of 1.5 feet per second can be assumed for the average terrace channel. Runoff soon

concentrates from sheet flow into small gullies. Their path of flow and location may change from one flood to the next. Ordinary tillage operations may obliterate them after each period of runoff. Still larger gullies are formed which under a good conservation practice are transformed into permanent grassed waterways.

The travel time (T_t) for each type of upland flow can be computed using equation 15.1. The summation of these travel times will equal the T_c in the upland or subwatershed, to the watershed outlet, or down to the point where hydraulic cross sections have been made for the stream hydraulics method.

In a small watershed the elapsed time for overland flow in figure 15.2 may be a substantial percent of the total watershed time of concentration. Conversely, it is a much smaller portion of the total time of concentration in larger watersheds. In watersheds larger than 2000 acres, it can usually be ignored by extrapolating the average measured velocity over the entire hydraulic distance as previously described.

The upland method should be limited to small watersheds (2000 acres or less) and to the sub-watershed portions of larger watersheds above and beyond the point where it is impractical to survey cross sections and make other detailed hydraulic measurements. This upstream limit is usually selected where natural reach storage ceases to be an important element in shaping a unit hydrograph for the watershed in question.

<u>Curve number method</u>

This method was developed for areas of less than 2000 acres.

Equation 15.4 was developed from research watershed data:

$$L = \frac{\ell^{0.8}\ (S+1)^{0.7}}{1900\ Y^{0.5}} \quad \text{. Eq. 15.4}$$

Where L = lag in hours

ℓ = hydraulic length of watershed in feet

$S = \dfrac{1000}{CN'} - 10$ where $CN' \cong$ hydrologic soil cover complex number (CN) in Chapter 9.

Y = average watershed land slope in percent

The curve number method was developed to span a broad set of conditions ranging from heavily forested watersheds with steep channels and a high percent of the runoff resulting from subsurface or inter-flow and meadows providing a high retardance to surface runoff, to smooth land surfaces and large paved parking areas. The CN' is a measure of the

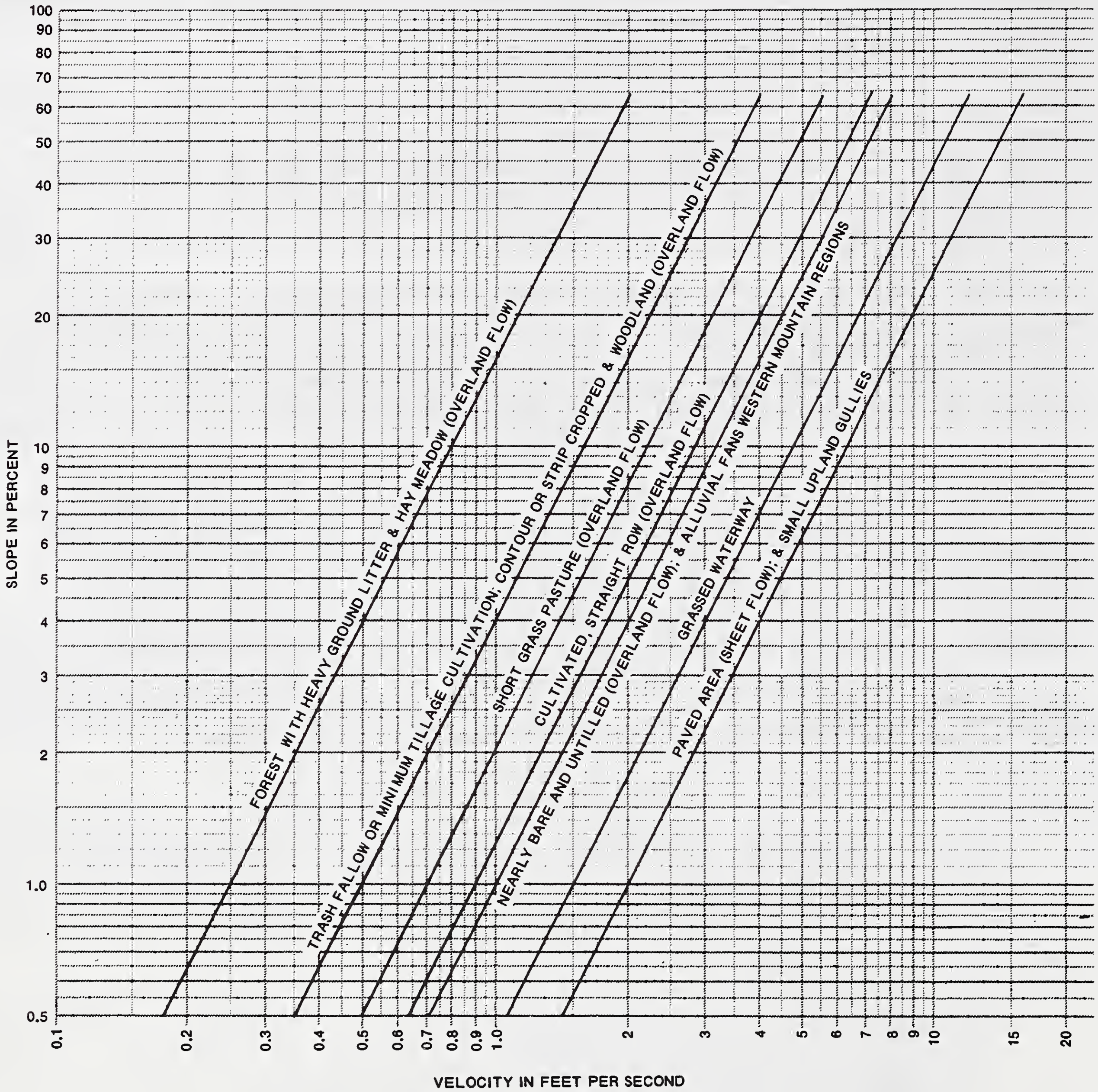

Figure 15.2.--Velocities for upland method of estimating T_c

retardance of surface conditions on the rate at which runoff concentrates at some point in question. This retardance factor (CN') is approximately the same as the CN in Chapter 9. A thick mulch in a forest is associated with a low CN in Chapter 9 and reflects a high degree of retardance as well as a high infiltration rate. A hay meadow has a relative low CN, other factors being equal, and like a thick mulch in a forest provides a high degree of retardance to overland flow in small watersheds. Conversely, bare surfaces with very little retardance to overland flow are represented by a high CN'. Runoff curve number tables in Chapter 9 can be used for approximating the CN' for the "S" in equation 15.4. A CN' of less than 50 or greater than 95 should not be used in the solution of equation 15.4.

The slope (Y) in percent is the average land slope of the watershed. Theoretically, it would be as if slopes were obtained for each corner of a grid system placed over the watershed, and then averaged.

Figure 15.3 provides a quick solution to equation 15.4.

Variations in Lag and T_c Due to Urbanization

Investigations have indicated that a significant increase in peak discharge can result from urbanization of a watershed. Such increases in the peak discharge are generally attributed to the construction of collection systems that are more efficient in a hydraulic sense than those provided in nature. These systems increase conveyance velocities so that greater amounts of discharge tend to reach points of concentration concurrently. Where flow once prevailed over a rough terrain and along field gullies and stream channels, urbanization provides hydraulically smooth concrete gutters, streets, storm drains and open channel floodways that convey runoff rapidly to downstream points.

The amount of imperviousness due to urbanization in a watershed varies from about 20 percent in the case of low density residential areas to about 90 percent where business and commercial land use predominates.

Table 15.1 illustrates the degree of imperviousness with land use for typical urban development.

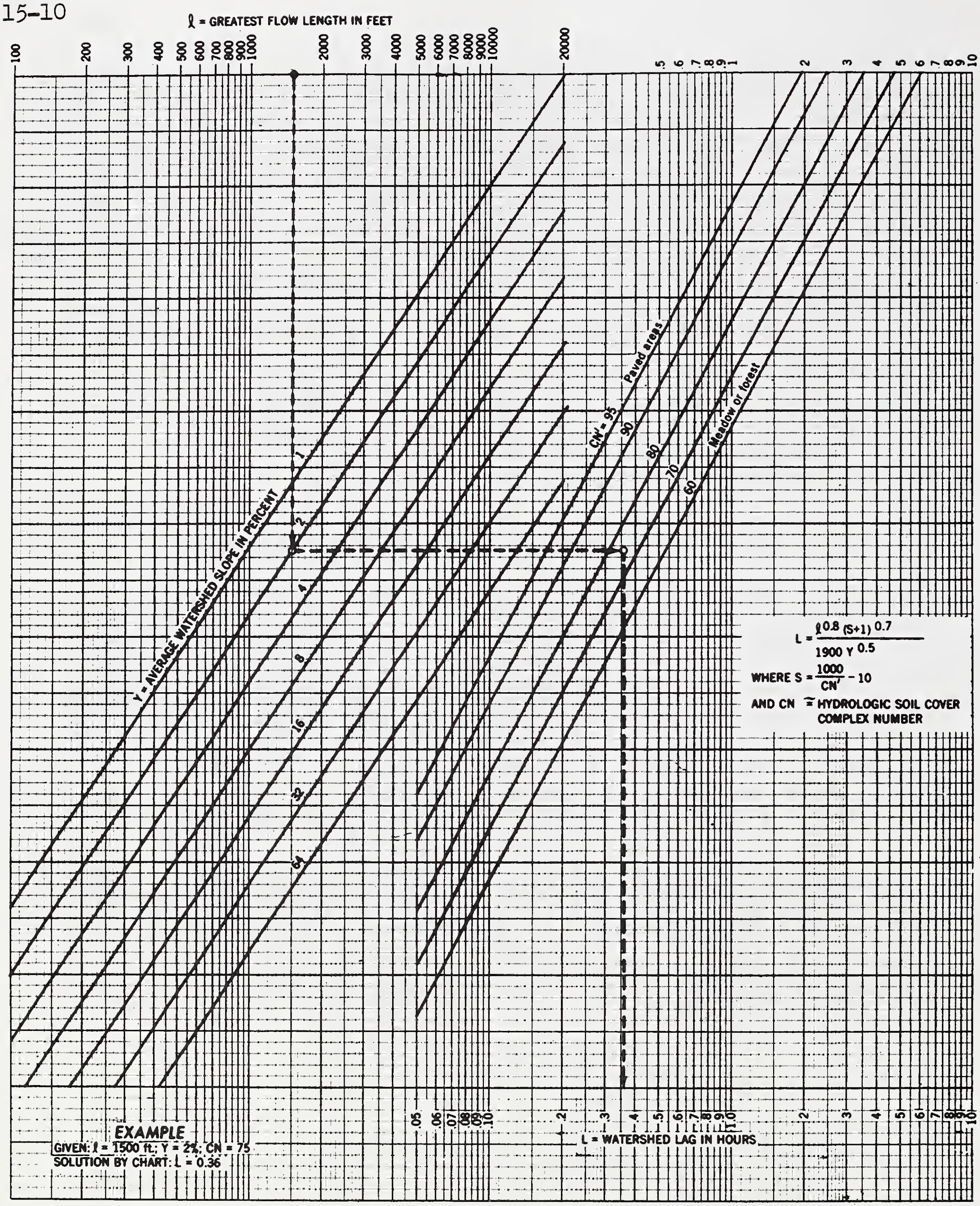

Figure 15.3.--Curve number method for estimating lag (L)

Table 15.1.--Percent of imperviousness for various densities of urban occupancy.

Land Use	% Imperviousness[1/]
Low Density Residential	20 - 30
Medium Density Residential	25 - 35
High Density Residential	30 - 40
Business - Commercial	40 - 90
Light Industrial	45 - 65
Heavy Industrial	50 - 70

1/ Effects of Urbanization on Storm Runoff - Cudworth and Bottorf - South Pacific Division - Corps of Engineers. Presented to Water Management Subcommittee, PSIAC, March 1969.

A CN' of 90 or 95 can be used to estimate the impervious portion. CN' for lawns, parks, etc. can be selected from one of the curve number tables in Chapter 9.

Travel Time Through Reservoirs, Lakes, and Swamps

It is sometimes necessary to compute a T_c for a watershed having a relatively large body of water in the flow path. In such cases, T_c is computed by one of the above methods to the upstream end of the lake or reservoir, and for the body of water the travel time is computed using the equation:

$$V_w = \sqrt{gD_m} \qquad \text{Eq. 15.5}$$

Where V_w = the wave velocity, in fps, across the water

g = 32.2 feet/sec/sec

D_m = mean depth of lake or reservoir in feet

Generally, V_w will be high, as shown in table 15.2.

One must not overlook the fact that equation 15.5 only provides for estimating travel time across the lake and for the inflow hydrograph to the lake's outlet. It does not account for the travel time involved with the passage of the inflow hydrograph through spillway storage and the reservoir or lake's outlet. This time is generally much longer than and is added to the travel time across the lake. The travel time through lake storage and its outlet can be determined by one of the storage routing procedures in Chapter 17.

For additional discussion of equation 15.5 see King's "Handbook of Hydraulics," fourth edition, page 8-50, or "Elementary Mechanics of Fluids" by Hunter Rouse, John Wiley and Sons, Inc., 1946, page 142.

Equation 15.5 can be used for swamps with much open water, but where the vegetation or debris is relatively thick (less than about 25 percent open water), Manning's equation is more appropriate.

Table 15.2.--Wave velocities on lakes and reservoirs

Mean depth, D_m (feet)	Wave velocities, V_w (fps)	(mph)
2	8.0	5.45
4	11.3	7.70
8	16.0	10.9
16	22.7	15.5
32	32.1	21.9

Examples

The following examples illustrate the use of the methods previously described to estimate travel time (T_t), time of concentration (T_c) and lag (L). The sample watershed of Chapter 6 showing the subdivision of a hydrologic unit is repeated here as figure 15.4 for the examples that follow.

Example 15.1, Upland Method.--Subdivision (1) in figure 15.4 has a diversion terrace below a short grass pasture outletting into a grassed waterway down to a road crossing. The overland flow length across the pasture down to the diversion terrace is 900 feet.

The length of the longer diversion terrace is 2100 feet. The average slope of the pasture is 8 percent. The grassed waterway is 2400 feet long with an average slope of 4 percent. A raw gully extends from the road crossing where the grassed waterway terminates, down to the point where a grade stabilization structure is planned. The length of the gully is 2700 feet with a 3 percent grade.

1. Read the following velocities from figure 15.2:

   Short grass pasture @ 8 percent . . . . 2 ft./sec.
   Grassed waterway @ 4 percent . . . . 3 ft./sec.
   Gully @ 3 percent . . . . 3.5 ft./sec.

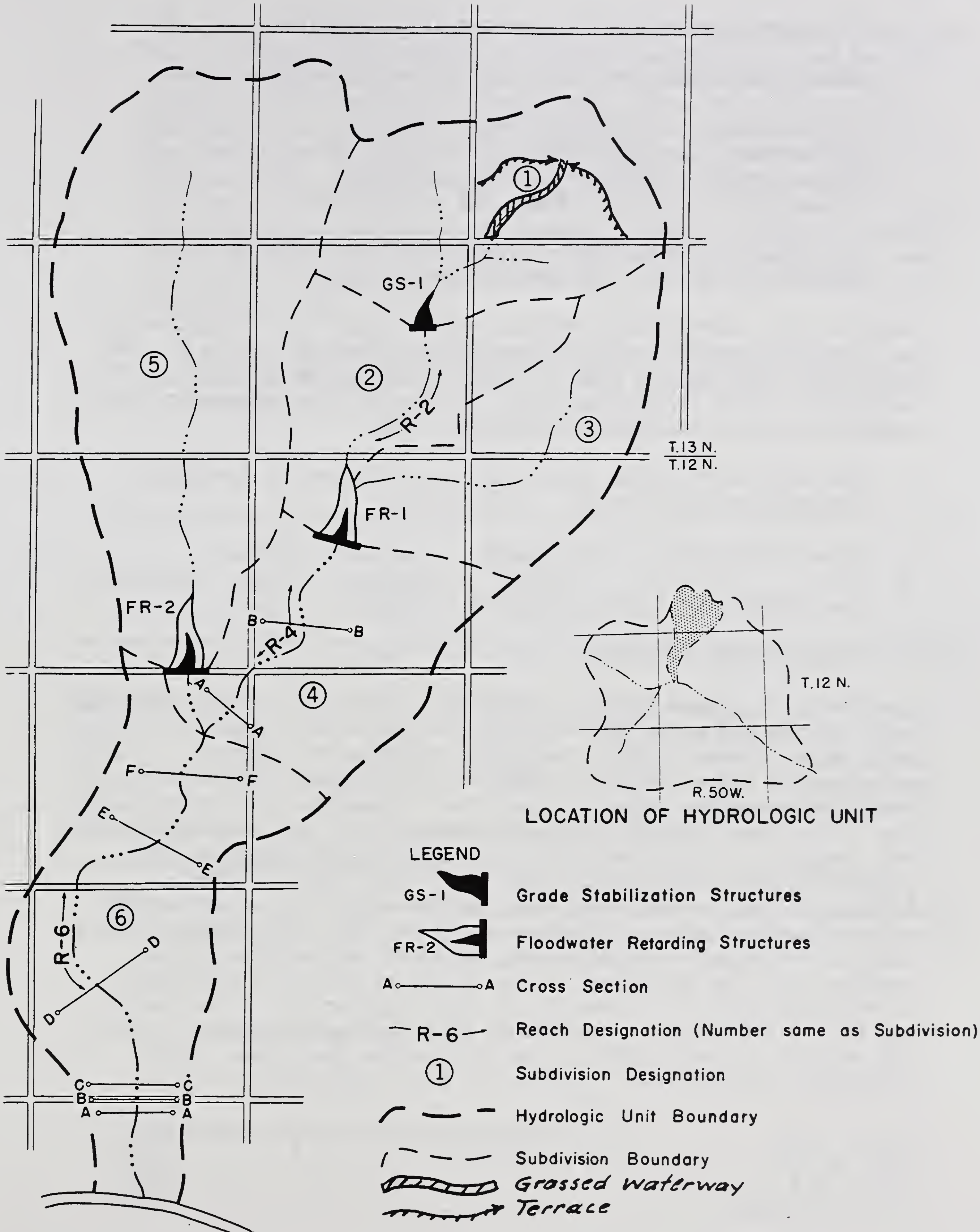

FIGURE 15.4-HYDROLOGIC UNIT HAVING DETAIL FOR USE AS A SAMPLE WATERSHED

2. The average velocity for terraces is 1.5 ft./sec.

3. Substituting velocity and length in equation 15.1:

   T_t (pasture) = (900/3600) ÷ 2 = 0.125 hr.
   T_t (terrace) = (2100/3600) ÷ 1.5 = 0.390 hr.
   T_t (waterway) = (2400/3600) ÷ 3 = 0.222 hr.
   T_t (gully) = (2700/3600) ÷ 3.5 = 0.215 hr.

4. $T_c = \Sigma T_t = 0.125 + 0.390 + 0.222 + 0.215 = 0.952$ hr.
   Round to 1.0 hr (to the nearest tenth hour).

Example 15.2, Curve Number Method.--Subdivision (5) in figure 15.4 is a wooded area with soils primarily in hydrologic group B. The hydrologic condition is good, having a heavy cover of litter. The slopes are steep, averaging about 16 percent. The hydraulic length according to map measurement is 16,000 feet.

1. The soil cover number from table 9-1 (Chapter 9) for this subdivision would be 55. $CN \cong CN' = 55$

2. Using figure 15.3, L = 1.4 hrs.

3. Use equation 15.3 to convert lag to T_c:

   $T_c = 1.4/0.6 = 2.3$ hrs.

Example 15.3, Stream Hydraulics Method.--It can be assumed that back water curves (or water surface profiles) have been computed by methods in Chapter 14 from the river outlet of the sample watershed in figure 15.4 up stream to the proposed floodwater retarding structure sites FR-1 and FR-2. Example 15.2 provided the T_c for developing inflow hydrographs to the proposed FR-2 site and example 15.1 provided the T_c for inflow hydrographs to the proposed grade stabilization structure, GS-1 site. A flood hydrograph for present conditions (without structures) is desired at the junction below subdivisions (4) and (5). Therefore a simple flood hydrograph is needed at the outlet of subdivision (4) to combine with the hydrograph at the proposed FR-2 site and outlet of subdivision (5). To develop a simple hydrograph at the lower end of subdivision (4), the travel time (T_t) is needed for reaches R-2 and R-4 and each added to the T_c for the GS-1 site. There floodplain lengths are:

GS-1 to FR-1 . . . . 6000'
FR-1 to B-B . . . . 2400'
B-B to A-A . . . . 2800'
A-A to junction . . . . 900'

The bankfull discharge and cross sectional area obtained from the W.S. profile rating curves at surveyed sections A-A and B-B give a mean velocity of 3.6 and 3.8 feet per second respectively. Similarly, the velocity obtained from the water surface profile at the FR-1 site is 6.1 feet per second. A surveyed cross section was available at the GS-1 site but other than that surveyed cross sections were not made beyond the upstream point of site FR-1. They were not considered necessary for the sole purpose of estimating travel time in this upper reach. Instead, handlevel channel cross sections were made at four intermediate locations in reach R-2 and an overall gradient estimated. These data appear in the following steps.

1. A table is made showing the field data obtained in R-2 and the estimated mean velocities for each section therein computed from Manning's formula, $V = \frac{1.486}{n} r^{2/3} S^{1/2}$

X-Section	Bankfull area (a)	Wetted Perimeter (P)	Hydraulic Radius (r)	$r^{2/3}$	n	$S^{1/2}$	V
	ft	ft	ft			ft/ft	ft/sec
GS-1	48	22	2.18	1.68	0.040	0.10	6.2
hde-1	55	35	1.57	1.35	0.055	0.10	3.7
hde-2	55	39	1.42	1.26	0.055	0.10	3.4
hde-3	50	26	1.92	1.55	0.040	0.10	5.8
hde-4	56	28	2.00	1.59	0.040	0.10	5.9
FR-1	(obtain from water surface profile rating)						6.1

2. Since the handlevel sections were taken at approximately equal intervals, the velocities are averaged without weighting them with respect to length. The average velocity for reach R-2 is 5.2 ft/sec.

3. Applying equation 15.1:

$$T_t = (6000/3600) \div 5.2 = 0.32 \text{ hrs.}$$

4. Obtain T_t for R-4 by equation 15.2:

From	To	Distance (d)	Velocity (V)	T_t (hr)
FR-1	Midway to B-B	1200	6.1	0.051
Midway between FR-1 & B-B	Midway between B-B & A-A	2600	3.8	0.190
Midway between B-B & A-A	junction	2300	3.6	0.181
			Total	0.422

5. T_c for subdivisions (1), (2), (3) and (4):

T_c for subdivision (1) from example 15.1 . . . . . 0.95
T_t for R-2 . . . . 0.32
T_t for R-4 . . . . 0.42
T_c (total) 1.69
Round to 1.7 hrs.

A hydrograph developed at the junction by combining the two tributary areas and using the longer T_c of 2.3 hours would be less accurate than by estimating the T_c for each tributary, as was done in the examples above, and then combining the two hydrographs developed for each.

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HYDROLOGY

CHAPTER 16. HYDROGRAPHS

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CHAPTER 16. HYDROGRAPHS

Purpose

Hydrographs, or some elements of them such as peak rates, are used in the planning and design of water control structures. They are also used to show the hydrologic effects of existing or proposed watershed projects.

Development of Hydrograph Relations

Runoff occurring on the uplands flows downstream in various patterns of flow which are affected by many factors such as spatial and temporal distribution of rainfall, rate of snowmelt, hydraulics of streams, watershed and channel storage, and others that are difficult to define. The graph of flow (rate versus time) at a stream section is the hydrograph, of which no two are exactly alike. There is no satisfactory mathematical analysis of flood hydrographs, and empirical relations have been developed, starting with the "Rational Method" in the 19th century, progressing to the Unit Hydrograph in the 1930's, and to more recent use of Dimensionless or Index Hydrographs. The empirical relations are simple elements from which as complex a hydrograph may be made as needed.

Present-day difficulties with hydrograph development lie in the precise estimation of runoff from rainfall (chapter 10) and determination of paths of flow (chapter 15).

Types of Hydrographs

This classification is a partial list, suitable for use in watershed work.

1. Natural hydrographs. Obtained directly from the flow records of a gaged stream.

2. Synthetic hydrographs. Obtained by using watershed parameters and storm characteristics to simulate a natural hydrograph.

3. Unit hydrograph. A natural or synthetic hydrograph for one inch of direct runoff. The runoff occurs uniformly over the watershed in a specified time.

4. Dimensionless hydrograph. Made to represent many unit hydrographs by using the time to peak and the peak rates as basic units and plotting the hydrographs in ratios of these units. Also called Index hydrograph.

Unit Hydrograph

In 1932, L.K. Sherman[1] advanced the theory of the unit hydrograph, or unit graph. The unit hydrograph procedure assumes that discharge at any time is proportional to the volume of runoff and that time factors affecting hydrograph shape are constant.

Both field data and laboratory tests have shown that the assumption of a linear relationship between watershed components is not strictly true. The non-linear relationships have not been investigated sufficiently to ascertain their effects on a synthetic hydrograph. Until more information is available the procedures of this chapter will be based on the unit hydrograph theory.

The fundamental principles of invariance and superposition make the unit graph an extremely flexible tool for developing synthetic hydrographs: 1) the hydrograph of surface runoff from a watershed due to a given pattern of rainfall is invariable, and 2) the hydrograph resulting from a given pattern of rainfall excess can be built up by superimposing the unit hydrograph due to the separate amounts of rainfall excess occurring in each unit period. This includes the principle of proportionality by which the ordinates of the hydrograph are proportional to the volume of rainfall excess.

The unit time or "unit hydrograph duration" is the optimum duration for occurrence of precipitation excess. In general, this unit time is approximately 20 percent of the time interval between the beginning of runoff from a short high-intensity storm and the peak discharge of the corresponding runoff.

The "storm duration" is the actual duration of the precipitation excess. The duration varies with actual storms. The dimensionless unit hydrograph used by SCS (figure 16.1) was developed by Victor Mockus. It was derived from a large number of natural unit hydrographs from watersheds varying widely in size and geographical locations. This dimensionless curvilinear hydrograph, also shown in table 16.1, has its ordinate values expressed in a dimensionless ratio q/q_p or Qa/Q and its abscissa values as t/T_p. This unit hydrograph has a point of inflection approximately 1.70 times the time-to-peak (T_p) and the time-to-peak 0.2 of the time-of-base (T_b).

[1]See References at end of chapter.

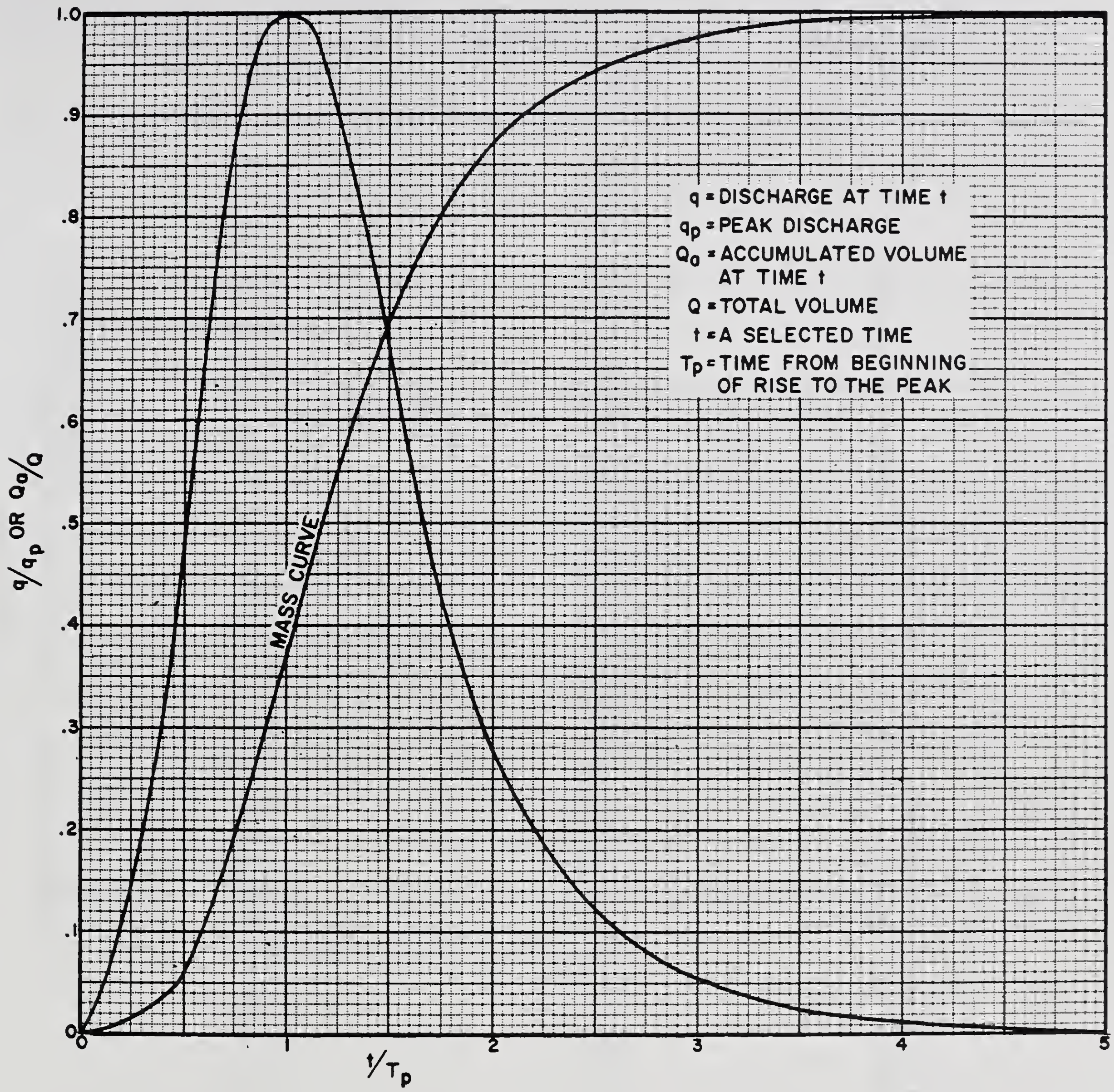

Figure 16.1 Dimensionless unit hydrograph and mass curve

Table 16.1 Ratios for dimensionless unit hydrograph and mass curve.

Time Ratios (t/T_p)	Discharge Ratios (q/q_p)	Mass Curve Ratios (Qa/Q)
0	.000	.000
.1	.030	.001
.2	.100	.006
.3	.190	.017
.4	.310	.035
.5	.470	.065
.6	.660	.107
.7	.820	.163
.8	.930	.228
.9	.990	.300
1.0	1.000	.375
1.1	.990	.450
1.2	.930	.522
1.3	.860	.589
1.4	.780	.650
1.5	.680	.705
1.6	.560	.751
1.7	.460	.790
1.8	.390	.822
1.9	.330	.849
2.0	.280	.871
2.2	.207	.908
2.4	.147	.934
2.6	.107	.953
2.8	.077	.967
3.0	.055	.977
3.2	.040	.984
3.4	.029	.989
3.6	.021	.993
3.8	.015	.995
4.0	.011	.997
4.5	.005	.999
5.0	.000	1.000

Elements of a Unit Hydrograph

The dimensionless curvilinear unit hydrograph (figure 16.1) has 37.5% of the total volume in the rising side, which is represented by one unit of time and one unit of discharge. This dimensionless unit hydrograph also can be represented by an equivalent triangular hydrograph having the same units of time and discharge, thus having the same percent of volume in the rising side of the triangle (figure 16.2).

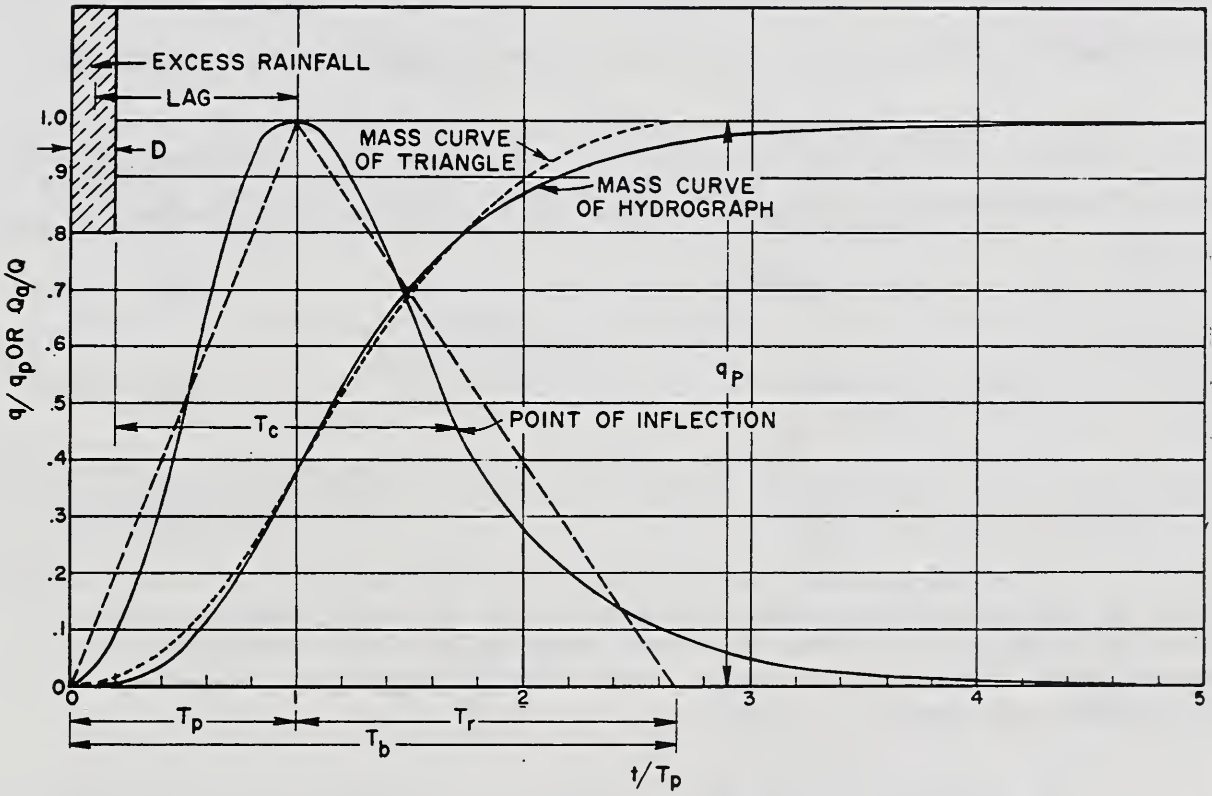

Figure 16.2 Dimensionless curvilinear unit hydrograph and equivalent triangular hydrograph

This allows the base of the triangle to be solved in relation to the time to peak using the geometry of triangles. Solving for the base length of the triangle, if one unit of time T_p equals .375 of volume:

$$T_b = \frac{1.00}{.375} = 2.67 \text{ units of time,}$$

$$T_r = T_b - T_p = 1.67 \text{ units of time or } 1.67\ T_p.$$

These relationships are useful in developing the peak rate equation for use with the dimensionless unit hydrograph.

Peak Rate Equation

From figure 16.2 the total volume under the triangular unit hydrograph is:

$$Q = \frac{q_p\ T_p}{2} + \frac{q_p\ T_r}{2} = \frac{q_p}{2}(T_p + T_r) \qquad \text{(Eq. 16.1)}$$

With Q in inches and T in hours, solve for peak rate q_p in inches per hour:

$$q_p = \frac{2Q}{T_p + T_r} \qquad \text{(Eq. 16.2)}$$

$$\text{Let}\quad K = \frac{2}{1 + \frac{T_r}{T_p}} \qquad \text{(Eq. 16.3)}$$

$$\text{Therefore}\quad q_p = \frac{KQ}{T_p} \qquad \text{(Eq. 16.4)}$$

In making the conversion from inches per hour to cubic feet per second and putting the equation in terms ordinarily used, including drainage area "A" in square miles, and time "T" in hours, equation 16.4 becomes the general equation:

$$q_p = \frac{645.33 \times K \times A \times Q}{T_p} \qquad \text{(Eq. 16.5)}$$

Where q_p is peak discharge in cubic feet per second (cfs) and the conversion factor 645.33 is the rate required to discharge one inch from one square mile in one hour.

The relationship of the triangular unit hydrograph, $T_r = 1.67\ T_p$, gives K = 0.75. Then substituting into equation 16.5 gives:

$$q_p = \frac{484\ A\ Q}{T_p} \qquad \text{(Eq. 16.6)}$$

Since the volume under the rising side of the triangular unit hydrograph is equal to the volume under the rising side of the curvilinear dimensionless unit hydrograph in figure 16.2, the constant 484, or peak rate factor, is valid for the dimensionless unit hydrograph in figure 16.1.

Any change in the dimensionless unit hydrograph reflecting a change in the percent of volume under the rising side would cause a corresponding change in the shape factor associated with the triangular hydrograph and therefore a change in the constant 484. This constant has been known to vary from about 600 in steep terrain to 300 in very flat swampy country. The E&WP Unit hydrologist should concur in the use of a dimensionless unit hydrograph other than figure 16.2. If for some reason it becomes necessary to vary the dimensionless shape of the hydrograph to perform a special job, the ratio of the percent of total volume in the rising side of the unit hydrograph to the rising side of a triangle is a useful tool in arriving at the peak rate factor.

Figure 16.2 shows that:

$$T_p = \frac{\Delta D}{2} + L \qquad \text{(Eq. 16.7)}$$

where ΔD is the duration of unit excess rainfall and L is the watershed lag in hours. The lag (L) of a watershed is defined (chapter 15) as the time from the center of mass of excess rainfall (ΔD) to the time to peak (T_p) of a unit hydrograph. From equation 16.6:

$$q_p = \frac{484\ A\ Q}{\frac{\Delta D}{2} + L} \qquad \text{(Eq. 16.8)}$$

The average relationship of lag (L) to time of concentration (T_c) is $L = 0.6\ T_c$ (chapter 15).

Substituting in equation 16.8, the peak rate equation becomes:

$$q_p = \frac{484\ A\ Q}{\frac{\Delta D}{2} + 0.6\ T_c} \qquad \text{(Eq. 16.9)}$$

The time of concentration is defined in two ways in chapter 15: 1) the time for runoff to travel from the furthermost point in the watershed to one point in question, and 2) the time from the end of excess rainfall to the point of inflection of the unit hydrograph.

These two relationships are important since T_c is computed under the first definition and ΔD, the unit storm duration, is used to compute the time to peak (T_p) of the unit hydrograph. This in turn is applied to all of the points on the abscissa of the dimensionless unit hydrograph using the ratio t/T_p as shown in table 16.1.

The dimensionless unit hydrograph shown in figure 16.2 has a time to peak at one unit of time and point of inflection at approximately 1.7 units of time. Using the relationships Lag = 0.6 T_c and the point of

inflection = 1.7 T_p, ΔD will be .2 T_p. A small variation in ΔD is permissible, however, it should be no greater than .25 T_p. See example 1.

Using the relationship shown on the dimensionless unit hydrograph, figure 16.2 to compute the relationship of ΔD to T_c:

$$T_c + \Delta D = 1.7\ T_p \qquad \text{(Eq. 16.10)}$$

$$\frac{\Delta D}{2} + .6\ T_c = T_p \qquad \text{(Eq. 16.11)}$$

Solving these two equations:

$$T_c + \Delta D = 1.7\ (\frac{\Delta D}{2} + .6\ T_c)$$

$$.15\ \Delta D = .02\ T_c$$

$$\Delta D = .133\ T_c \qquad \text{(Eq. 16.12)}$$

Application of Unit Hydrograph

The unit hydrograph can be constructed for any location on a uniformly shaped watershed, once the values of q_p and T_p are defined (figure 16.3, areas A and B).

Area C in figure 16.3 is an irregularly shaped watershed having two uniformly shaped areas (C2 and C1) with a big difference in their time of concentration. This watershed requires the development of two unit hydrographs which may be added together forming one irregularly shaped unit hydrograph. This irregularly shaped unit hydrograph may be used to develop a flood hydrograph in the same way as the unit hydrograph developed from the dimensionless form (figure 16.1) is used to develop the flood hydrograph. See example 1 for area shown in figure 16.3. Also, each of the two unit hydrographs developed for areas C2 and C1 in figure 16.3 may be used to develop a flood hydrograph for its respective C2 and C1 areas. The flood hydrographs from each area are then combined to form the hydrograph at the outlet of area C.

There are many variables integrated into the shape of a unit hydrograph. Since a dimensionless unit hydrograph is used and the only parameters readily available from field data are drainage area and time of concentration, consideration should be given to dividing the watershed into hydrologic units of uniformly shaped areas. These divisions, if at all possible, should be no greater than 20 square miles in area and should have a homogeneous drainage pattern.

The "storm duration" is the actual time duration of precipitation excess. This time duration varies with actual storms and should not be confused with the unit time or unit hydrograph duration.

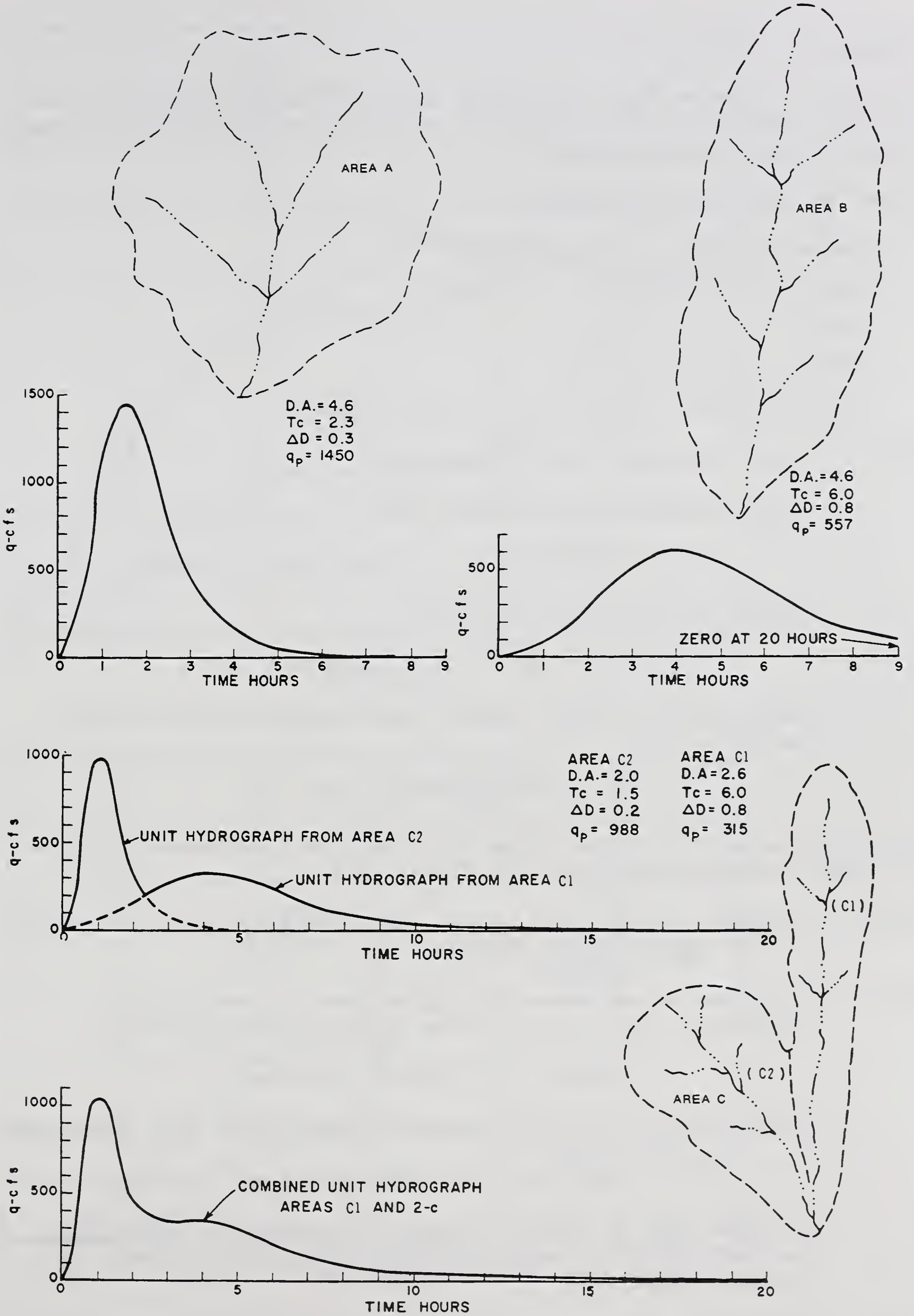

Figure 16.3 The effect of watershed shape on the peaks of unit hydrographs

Example 1

Develop a composite flood hydrograph using the runoff produced by the rainfall taken from a recording rain gage (figure 16.4(a)) on watershed (Area A) shown on figure 16.3.

Given the following information:

Drainage Area - 4.6 square miles

Time of Concentration - 2.3 hours

CN-85

Moisture Condition II

Storm Duration - 6 hours

Step 1. Develop and plot unit hydrograph.

Using equation 16.12, compute ΔD:

$$\Delta D = .133 \times 2.3 = .306 \quad \text{use .30 hours}$$

Using equation 16.7, compute T_p:

$$T_p = \frac{.30}{2} + (.6 \times 2.3) = 1.53 \text{ hours}$$

Using equation 16.6, compute q for volume of runoff equal to one inch:

$$q_p = \frac{484 \times 4.6 \times 1}{1.53} = 1450 \text{ cfs}$$

The coordinates of the curvilinear unit hydrograph are shown in table 16.2 and the plotted hydrograph on figure 16.5.

Step 2. Tabulate the ordinates of the unit hydrograph from figure 16.5 in 0.3 hour increments (table 16.3a, column 2).

Step 3. Check the volume under unit hydrograph by summing the ordinates (table 16.3a, column 2) and multiplying by ΔD:

$$9898 \times 0.3 = 2969.4 \text{ cfs-hours}$$

Compare this figure with computed volume under unit hydrograph:

$$645.33 \times 4.6 = 2968 \text{ cfs-hours}$$

If these fail to check, re-read the coordinates from figure 16.5 and adjust if necessary until a reasonable balance in volume is attained.

Step 4. Tabulate the accumulated rainfall in .3 hour increments (table 16.4, column 2).

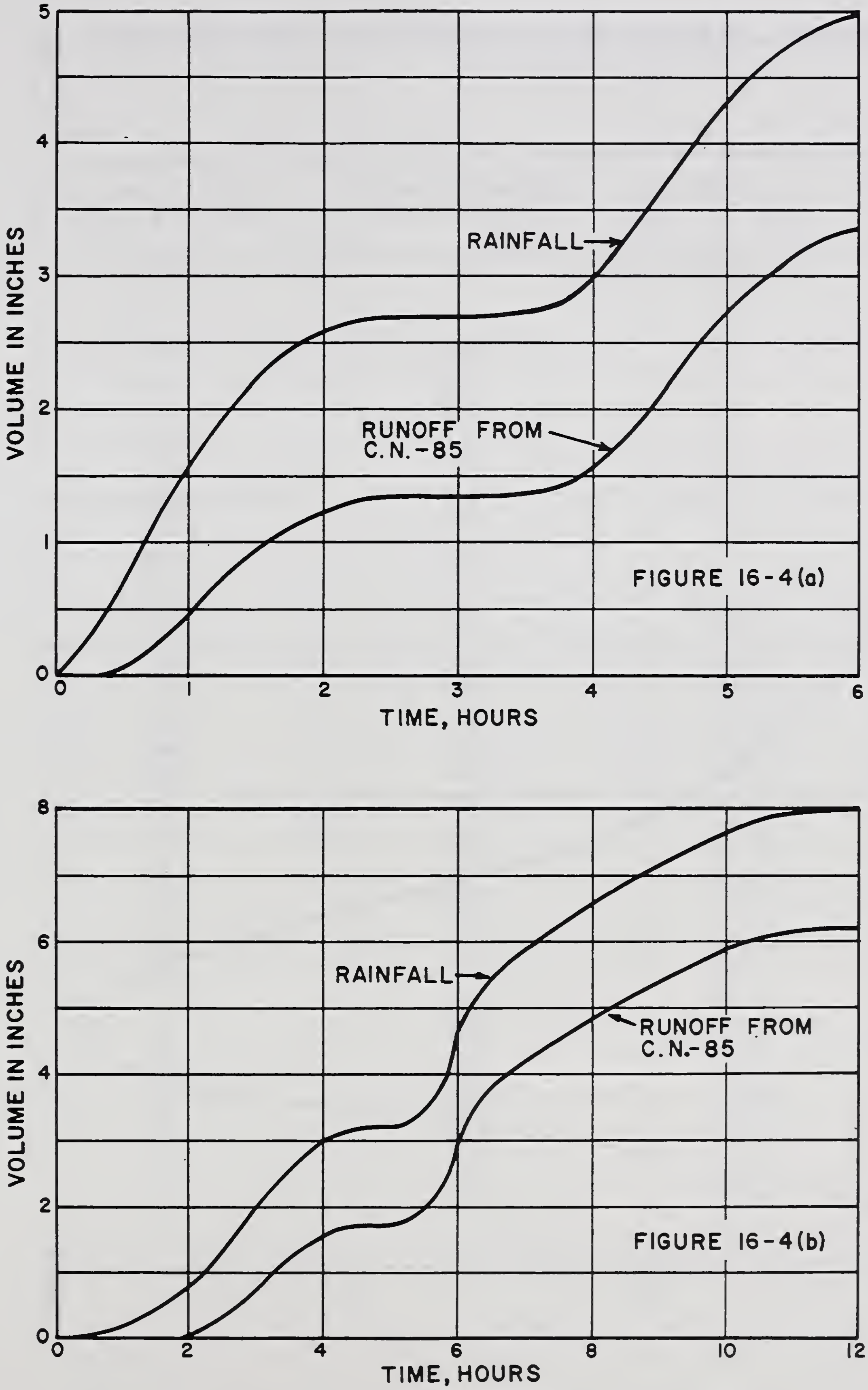

Figure 16.4 Accumulated rainfall and runoff for CN-85 taken from a recording rain gage.

Table 16.2. Computation of coordinates for unit hydrograph for use in Example 1.

1	2	3	4
Time Ratios (table 16.1)	Time (col 1 x 1.53)	Discharge Ratios (table 16.1)	Discharges (col 3 x 1450)
(t/T_p)	(hours)	(q/q_p)	(cfs)
.0	0	0	0
.1	.15	.030	44
.2	.31	.100	145
.3	.46	.190	276
.4	.61	.310	450
.5	.76	.470	682
.6	.92	.660	957
.7	1.07	.820	1189
.8	1.22	.930	1349
.9	1.38	.990	1435
1.0	1.53	1.000	1450
1.1	1.68	.990	1435
1.2	1.84	.930	1349
1.3	1.99	.860	1247
1.4	2.14	.780	1131
1.5	2.29	.680	986
1.6	2.45	.560	812
1.7	2.60	.460	667
1.8	2.75	.390	565
1.9	2.91	.330	479
2.0	3.06	.280	406
2.2	3.37	.207	300
2.4	3.67	.147	213
2.6	3.98	.107	155
2.8	4.28	.077	112
3.0	4.59	.055	80
3.2	4.90	.040	58
3.4	5.20	.029	42
3.6	5.51	.021	30
3.8	5.81	.015	22
4.0	6.12	.011	16
4.5	6.89	.005	7
5.0	7.65	0	0

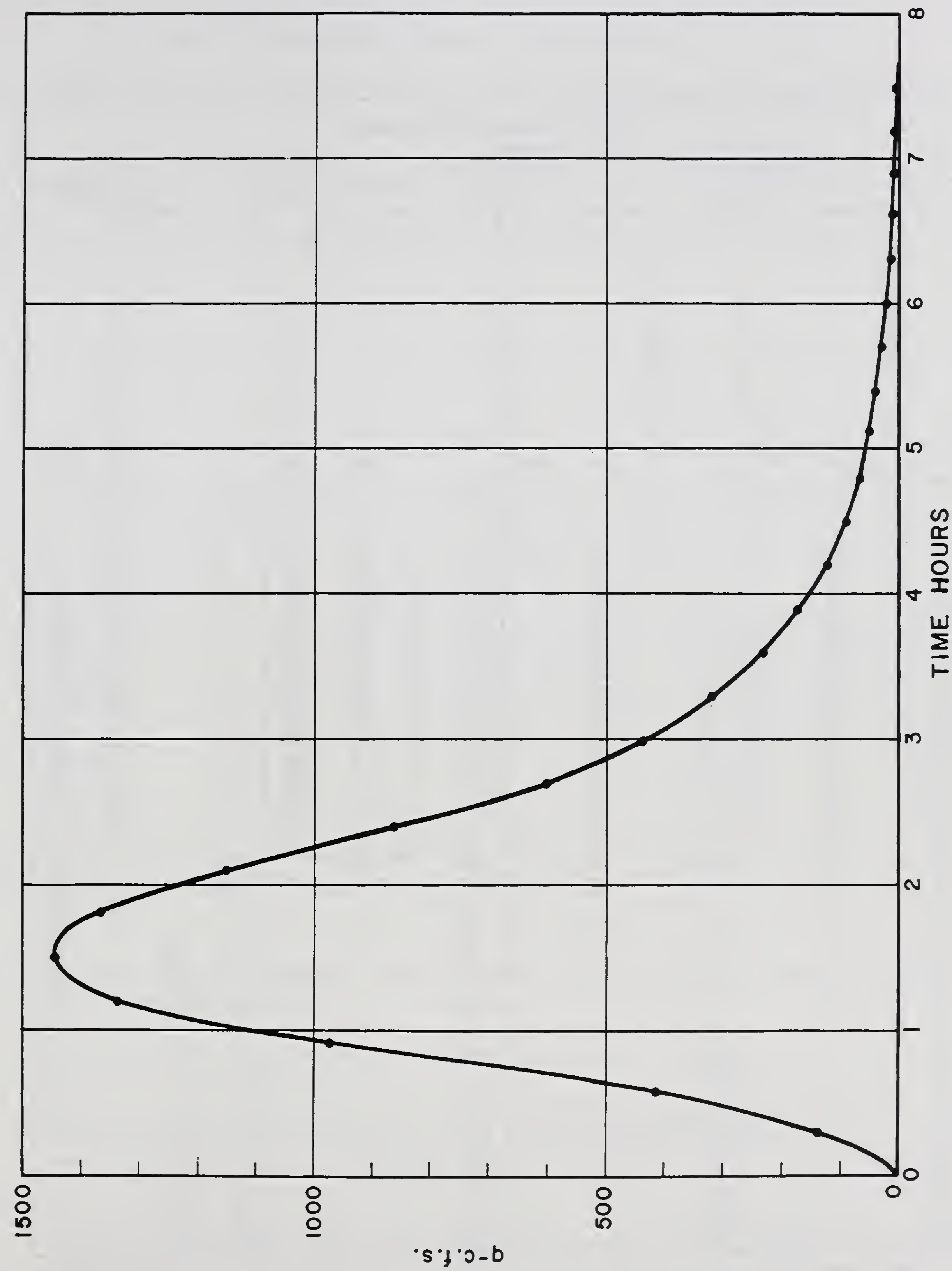

Figure 16.5 Unit hydrograph from example 1

Table 16.3. Computation of a flood hydrograph (example 1).

Table 16.3 (a)

(1) Time		(2) Unit Hyd.	(3) Flood Hyd.
	.09		
	.19		
	.24		
	.31		
	.42		
	.36		
	.25		
	.11		
	.05		
	.01		
	.0		
	.0		
	.06		
	.12		
	.18		
	.26		
	.33		
0	.27	0	0
.3	.12	140	0
.6	.0 →	420	17
.9		960	
1.2		1330	
1.5		1450	
1.8		1370	
2.1		1140	
2.4		860	
2.7		610	
3.0		440	
3.3		320	
3.6		230	
3.9		170	
4.2		120	
4.5		85	
4.8		70	
5.1		55	
5.4		40	
5.7		30	
6.0		20	
6.3		15	
6.6		10	
6.9		7	
7.2		4	
7.5		2	
7.8		0	
8.1			
8.4			
8.7			
9.0			
9.3			
9.6			
9.9			
10.2			
10.5			
10.8			
11.1			
11.4			
11.7			
12.0			
12.3			
12.6			
12.9			
13.2			
13.5			
13.7			

Total 9898

Table 16.3 (b)

(1) Time		(2) Unit Hyd.	(3) Flood Hyd.
	.09		
	.19		
	.24		
	.31		
	.42		
	.36		
	.25		
	.11		
	.05		
	.01		
	.0		
	.0		
	.06		
	.12		
	.18		
	.26		
0	.33	0	0
.3	.27	140	0
.6	.12	420	17
.9	.0 →	960	88
1.2		1330	
1.5		1450	
1.8		1370	
2.1		1140	
2.4		860	
2.7		610	
3.0		440	
3.3		320	
3.6		230	
3.9		170	
4.2		120	
4.5		85	
4.8		70	
5.1		55	
5.4		40	
5.7		30	
6.0		20	
6.3		15	
6.6		10	
6.9		7	
7.2		4	
7.5		2	
7.8		0	
8.1			
8.4			
8.7			
9.0			
9.3			
9.6			
9.9			
10.2			
10.5			
10.8			
11.1			
11.4			
11.7			
12.0			
12.3			
12.6			
12.9			
13.2			
13.5			
13.7			

Table 16.3 (c)

(1) Time		(2) Unit Hyd.	(3) Flood Hyd.
0		0	0
.3		140	0
.6	.09	420	17
.9	.19	960	88
1.2	.24	1330	275
1.5	.31	1450	594
1.8	.42	1370	984
2.1	.36	1140	1337
2.4	.25	860	1563
2.7	.11	610	1620
3.0	.05	440	1516
3.3	.01	320	1300
3.6	.0	230	1050
3.9	.0	170	838
4.2	.06	120	726
4.5	.12	85	765
4.8	.18	70	988
5.1	.26	55	1359
5.4	.33	40	1787
5.7	.27	30	2143
6.0	.12	20	2342
6.3	.0 →	15	2350
6.6		10	
6.9		7	
7.2		4	
7.5		2	
7.8		0	
8.1			
8.4			
8.7			
9.0			
9.3			
9.6			
9.9			
10.2			
10.5			
10.8			
11.1			
11.4			
11.7			
12.0			
12.3			
12.6			
12.9			
13.2			
13.5			
13.7			

Table 16.3 (d)

(1) Time		(2) Unit Hyd.	(3) Flood Hyd.
0		0	0
.3		140	0
.6		420	17
.9		960	88
1.2		1330	275
1.5		1450	594
1.8		1370	984
2.1		1140	1337
2.4		860	1563
2.7		610	1620
3.0		440	1516
3.3		320	1300
3.6		230	1050
3.9		170	838
4.2		120	726
4.5		85	765
4.8		70	988
5.1		55	1359
5.4		40	1787
5.7		30	2143
6.0		20	2342
6.3		15	2350
6.6		10	2170
6.9		7	1854
7.2		4	1488
7.5		2	1138
7.8	.09	0	840
8.1	.19		608
8.4	.24		438
8.7	.31		318
9.0	.42		233
9.3	.36		172
9.6	.25		128
9.9	.11		96
10.2	.05		72
10.5	.01		53
10.8	.0		38
11.1	.0		27
11.4	.06		18
11.7	.12		12
12.0	.18		7
12.3	.26		4
12.6	.33		2
12.9	.27		1
13.2	.12		0
13.5	.0 →		
13.7			

Total 33359

Table 16.4 Rainfall tabulated in 0.3 hour increments from plot of Rain Gage Chart, Figure 16.4a

1 Time	2 Accum. Rainfall	3 Accum.[1] Runoff	4 Incremental Runoff	5 Reversed Incremental Runoff
0	0			
.3	.37	.00	00	.09
.6	.87	.12	.12	.19
.9	1.40	.39	.27	.24
1.2	1.89	.72	.33	.31
1.5	2.24	.98	.26	.42
1.8	2.48	1.16	.18	.36
2.1	2.63	1.28	.12	.25
2.4	2.70	1.34	.06	.11
2.7	2.70	1.34	.00	.05
3.0	2.70	1.34	.00	.00
3.3	2.71	1.35	.01	.00
3.6	2.77	1.40	.05	.00
3.9	2.91	1.51	.11	.06
4.2	3.20	1.76	.25	.12
4.5	3.62	2.12	.36	.18
4.8	4.08	2.54	.42	.26
5.1	4.43	2.85	.31	.33
5.4	4.70	3.09	.24	.27
5.7	4.90	3.28	.19	.12
6.0	5.00	3.37	.09	.00

[1]Runoff computed using CN 85 moisture condition II.

Step 5. Compute the accumulated runoff (table 16.4, column 3) using CN-85, condition II.

Step 6. Tabulate the incremental runoff (table 16.4, column 4).

Step 7. Tabulate the incremental runoff <u>in reverse order</u> (table 16.4, column 5) and/or tabulate it on a strip of paper having the same line spacing as the paper used in step 2.

Step 8. Place the strip of paper between column 1 and column 2 of table 16.3(a) and slide down until the first increment of runoff (0.12) on the strip of paper is opposite the first discharge (140) on the unit hydrograph (column 2). Multiplying 0.12 x 140 = 16.8 (round to 17). Tabulate in column 3 opposite the arrow on the strip of paper.

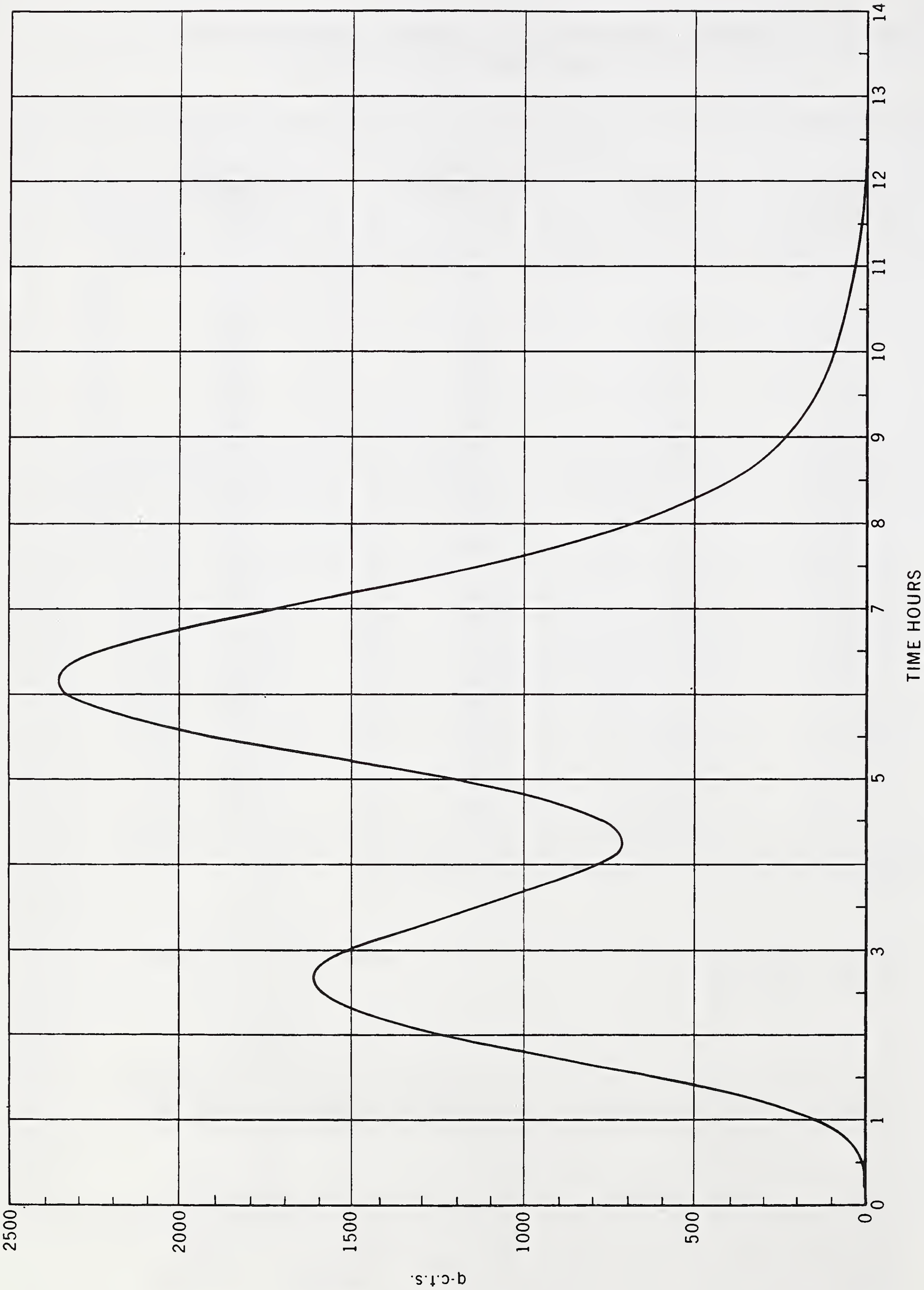

Figure 16.6 Composite flood hydrograph from example 1

Step 9. Move the strip of paper down one line (table 16.3(b)) and compute (0.12 x 420) + (.27 x 140) = 88.2 (round to 88). Tabulate in column 3 opposite the arrow on the strip of paper.

Continue moving the strip of paper containing the runoff down one line at a time and accumulatively multiply each runoff increment by the unit hydrograph discharge opposite the increment.

Table 16.3(c) shows the position of the strip of paper containing the runoff when the peak discharge of the flood hydrograph (2350 cfs) is reached. If only the peak discharge of the flood hydrograph is desired, it can be found by making only a few computations, placing the larger increments of runoff near the peak discharge of the unit hydrograph.

Figure 16.3(d) shows the position of the strip of paper containing the runoff at the completion of the flood hydrograph. The complete flood hydrograph is shown in column 3. These discharges are plotted at their proper time sequence on figure 16.6 which is the complete flood hydrograph for example 1.

Step 10. Check the volume under the flood hydrograph by summing the ordinates (table 16.3(d), column 3) and multiplying by ΔD. 33359 x .3 = 10007.7 cfs-hours, compared to computed volume, 645.33 x 4.6 x 3.37 = 10003.9 cfs-hours.

Example 2

Using the same data given in example 1, graphically develop a composite flood hydrograph using a triangle for the unit hydrograph.

Step 1. Plot the triangular unit hydrograph (dashed line) on figure 16.7: T_p = 1.53 hours, T_b = 4.08 hours.

Step 2. Compute the peak discharge for the first incremental triangular hydrograph by multiplying the first increment of runoff shown in table 16.4, column 4, by the peak discharge for one inch of runoff (1450). The peak of the first incremental triangular hydrograph is 1450 x .12 = 174. Since the storm did not produce runoff for the first increment of time and the zero point of the first incremental triangular hydrograph is plotted at 0.3 hours. The peak discharge of 174 cfs is plotted at 1.83 hours and end of the base is 4.38 hours. Continue developing and plotting incremental triangular hydrographs for each increment of runoff shown in table 16.4, column 4. Each incremental hydrograph is plotted one ΔD (0.3) hour later in time.

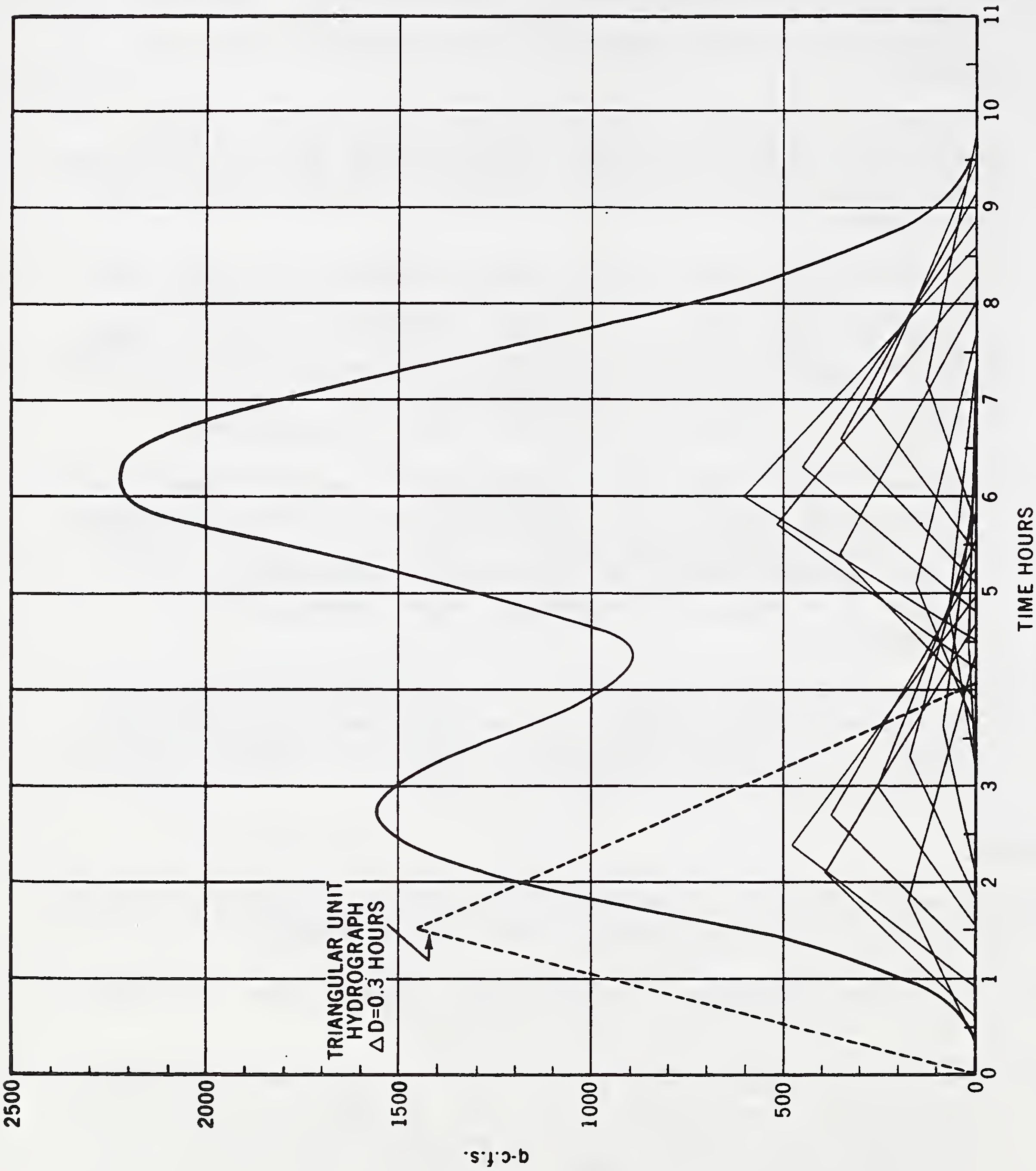

Figure 16.7 Composite flood hydrograph from example 2

Step 3. Sum the ordinates of each incremental triangular hydrograph at enough locations to make it possible to draw the completed flood hydrograph (figure 16.7). The composite peak is 2230 cfs.

Step 4. Check the area under the completed hydrograph and convert to cfs/hours, which is 40 sq. inches x 250 cfs-hours/sq. inch = 10,000 cfs-hours compared to the computed volume 645.33 x 4.6 x 3.37 = 10,003.9 cfs-hours. (Note: figure 16.7 has been reduced.)

Example 3

Using the same data given in example 1, but using a ΔD of 1.5 hours rather than 0.3 hour, graphically develop a composite flood hydrograph produced by the runoff from the rainfall shown on figure 16.4(a) and tabulated in table 16.5, column 4. This example will illustrate the effect of using a ΔD which is too large.

$$T_p = \frac{1.5}{2} + (.6 \times 2.3) = 2.13 \text{ hours}$$

$$T_b = 2.13 \times 2.67 = 5.68 \text{ hours}$$

$$q_p = \frac{484 \times 4.6 \times 1}{2.13} = 1043 \text{ cfs}$$

Following the same procedure outlined in example 2 of computing, plotting, and summing the ordinates of the incremental triangular hydrographs, a composite flood hydrograph is developed as shown in figure 16.8.

Table 16.5 Rainfall tabulated in 1.5 hour increments from plot of Rain Gage Chart, Figure 16.4a

Time	Accum. Rainfall	Accum. Runoff[1]	Incremental Runoff
0	0		
1.5	2.24	.98	.98
3.0	2.70	1.34	.36
4.5	3.62	2.12	.78
6.0	5.00	3.37	1.25

[1]Runoff computed using CN-85 moisture condition II.

The area under the composite flood hydrograph should be determined and the volume checked against the computed volume.

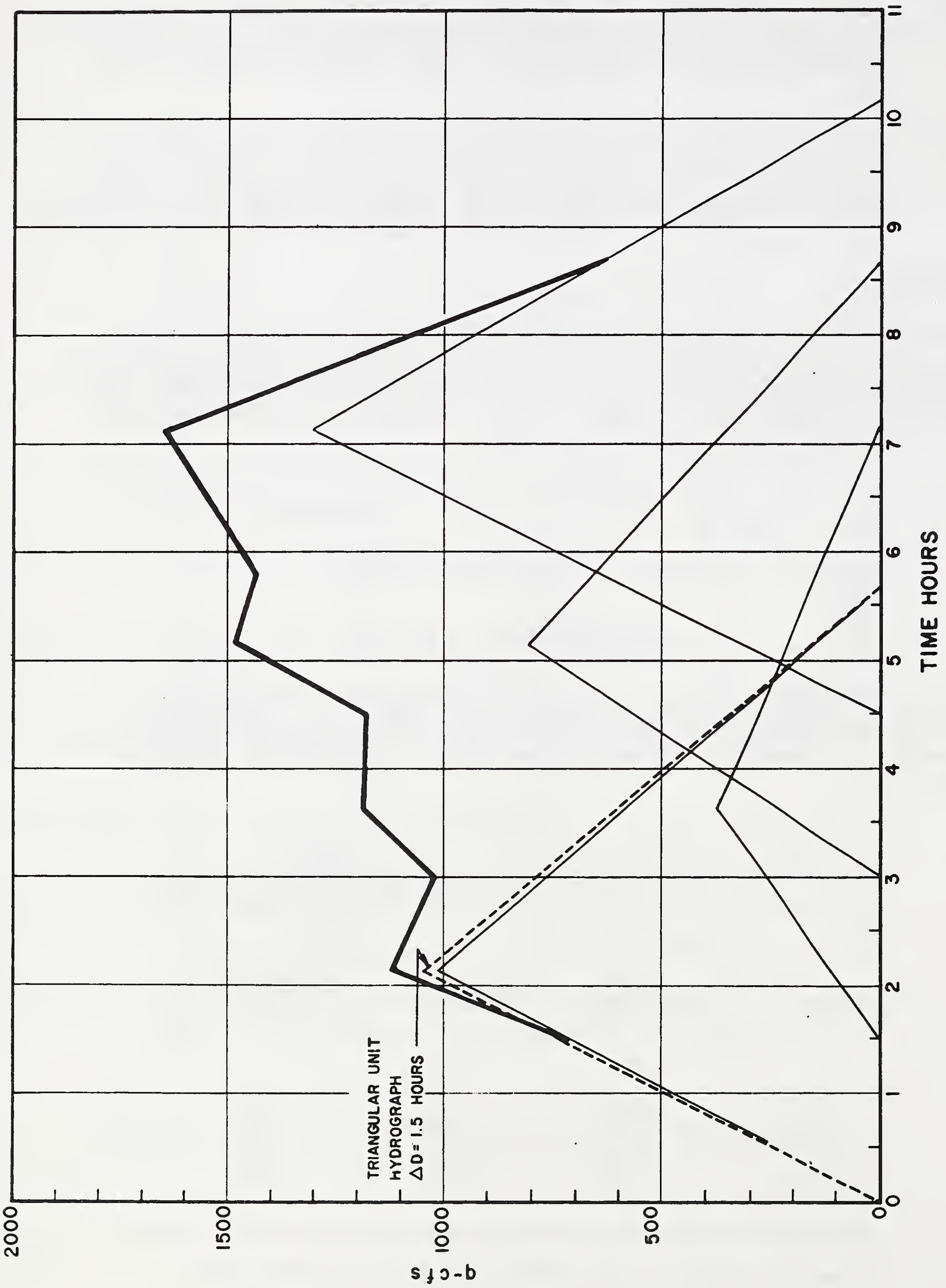

Figure 16.8 Composite flood hydrograph from example 3 showing effect when ΔD is too large

Examples 1 and 2 show that there is very little difference in the flood hydrograph developed using either a curvilinear unit hydrograph or a triangular unit hydrograph providing the unit of time (ΔD) is approximately 0.2 the time to peak of the unit hydrograph. This is the time defined by Mitchell, 1948, as the optimum time of a unit storm. Example 3 shows the effect of increasing the time increment to 1.5 hours which is approximately equal to the time to peak of the unit hydrograph when the optimum time increment is used.

Peak Discharge Determination

In using the triangular unit hydrograph to develop composite flood hydrographs, the peak of each triangular unit hydrograph is determined by multiplying the peak for one inch of runoff by the amount of runoff in each ΔD time. Assuming uniform runoff for an indefinite period of time and using ΔD as 0.2 of the time to peak of the unit hydrograph, figure 16.9 shows that 13 increments of runoff is the maximum number that will add to the peak discharge of the flood hydrograph. It also shows the percent of the peak of each incremental hydrograph that contributes to the peak of the composite flood hydrograph.

Table 16.7, column 2, shows a tabulation of these percentages in decimal form. This tabulation is used to compute the peak discharge and time to peak for any duration or pattern of rainfall.

Example 4

Compute the peak discharge and time to peak produced by the runoff from the rainfall shown in figure 16.4(b) and Table 16.6 for two locations on a homogeneous watershed. Given the following information:

Location 1 - Drainage Area, 2 square miles; T_c - 1.5 hours; CN-85.

Location 2 - Drainage Area, 20 square miles; T_c - 6 hours; CN-85; storm duration - 12 hours.

For Location 1:

Step 1. Compute the time increment ΔD.

From equation 16.12: $\Delta D = .133 \times 1.5 = .2$ hour

Step 2. Compute q_p the peak discharge for the unit hydrograph.

From equation 16.9:

$$q_p = \frac{484 \times 2 \times 1}{\frac{.2}{2} + .9} = 968 \text{ cfs}$$

Step 3. Knowing that 13 ΔD's is the maximum number of runoff increments that will contribute to the peak of the flood hydrograph, compute the maximum length of excess rainfall or runoff that will

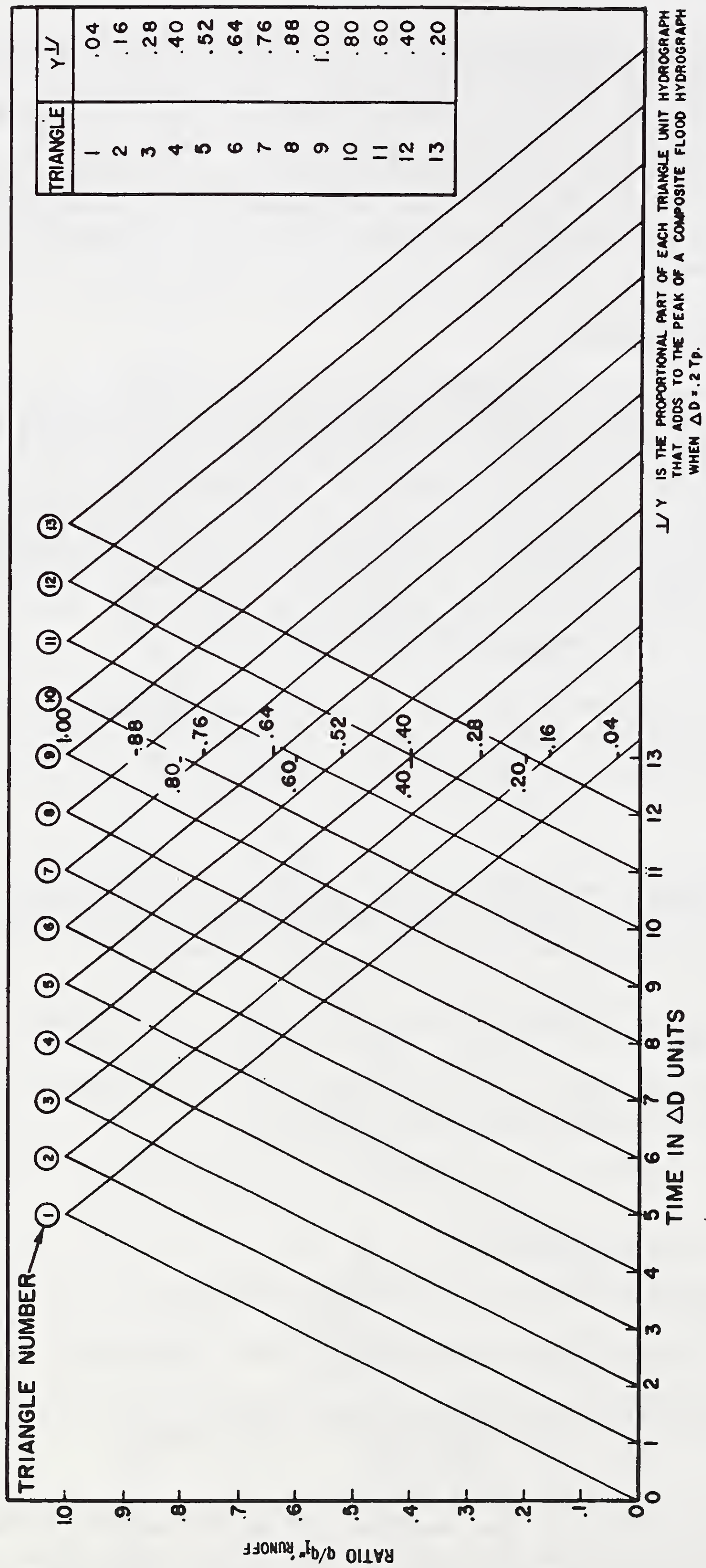

Figure 16.9 Part of triangular unit hydrograph that contributes to the peak when $\Delta D = 0.2\ T_p$

Table 16.6 Rainfall tabulated in 0.2 hour increments from plot of Rain Gage Chart, Figure 16.4(b)

Time (hours) (1)	Accum. Rainfall (in.) (2)	Accum.1/ Runoff (in.) (3)	ΔQ (in.) (4)	Time (hours) (1)	Accum. Rainfall (in.) (2)	Accum.1/ Runoff (in.) (3)	ΔQ (in.) (4)
0	0	0	0	6.2	5.00	3.37	.41
.2	.02	0	0	6.4	5.35	3.70	.33
.4	.05	0	0	6.6	5.52	3.86	.16
.6	.08	0	0	6.8	5.68	4.01	.15
.8	.13	0	0	7.0	5.83	4.15	.14
1.0	.20	0	0	7.2	6.00	4.31	.16
1.2	.27	0	0	7.4	6.15	4.45	.14
1.4	.36	0	0	7.6	6.30	4.59	.14
1.6	.48	0	0	7.8	6.42	4.71	.12
1.8	.60	.03	.03	8.0	6.54	4.82	.11
2.0	.80	.09	.06	8.2	6.66	4.93	.11
2.2	.95	.15	.06	8.4	6.80	5.06	.13
2.4	1.18	.27	.12	8.6	6.90	5.16	.10
2.6	1.45	.42	.15	8.8	7.02	5.28	.12
2.8	1.68	.58	.16	9.0	7.12	5.37	.09
3.0	2.00	.80	.22	9.2	7.21	5.46	.09
3.2	2.22	.96	.16	9.4	7.30	5.55	.09
3.4	2.42	1.12	.16	9.6	7.40	5.64	.09
3.6	2.62	1.27	.15	9.8	7.50	5.74	.10
3.8	2.82	1.43	.16	10.0	7.60	5.84	.10
4.0	3.00	1.59	.16	10.2	7.70	5.93	.09
4.2	3.10	1.68	.09	10.4	7.80	6.03	.10
4.4	3.18	1.74	.06	10.6	7.86	6.09	.06
4.6	3.20	1.76	.02	10.8	7.90	6.12	.03
4.8	3.20	1.76	.00	11.0	7.92	6.14	.02
5.0	3.21	1.77	.01	11.2	7.94	6.16	.02
5.2	3.23	1.79	.02	11.4	7.96	6.18	.02
5.4	3.38	1.91	.12	11.6	7.98	6.20	.02
5.6	3.60	2.11	.20	11.8	7.99	6.21	.01
5.8	3.83	2.31	.20	12.0	8.00	6.22	.01
6.0	4.55	2.96	.65				

1/ Runoff computed using CN = 85 moisture condition II

Table 16.7 Peak discharge determined for example 4.

Triangle Number	Y 1/	Location 1 ΔD = 0.2 hour			Location 2 ΔD = 0.8 hour Trial 1			Location 2 ΔD = 0.8 hours Trial 2		
		Time (hours)	Runoff (inches)	Col 2 x col 4	Time (hours)	Runoff (inches)	Col 2 x col 7	Time (hours)	Runoff (inches)	Col 2 x col 10
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)
1	.04	4.2	.06	.0024						
2	.16	4.4	.02	.0032	.0	.0		.8	.0	
3	.28	4.6	.00	.0000	.8	.0		1.6	.27	.0756
4	.40	4.8	.01	.0040	1.6	.27	.1080	2.4	.69	.2760
5	.52	5.0	.02	.0104	2.4	.69	.3588	3.2	.63	.3276
6	.64	5.2	.12	.0768	3.2	.63	.4032	4.0	.17	.1088
7	.76	5.4	.20	.1520	4.0	.17	.1292	4.8	.35	.2660
8	.88	5.6	.20	.1760	4.8	.35	.3080	5.6	1.59	1.3992
9	1.00	5.8 2/	.65	.6500	5.6 2/	1.59	1.5900	6.4 2/	.61	.6100
10	.80	6.0	.41	.3280	6.4	.61	.4880	7.2	.51	.4080
11	.60	6.2	.33	.1980	7.2	.51	.3060	8.0	.46	.2760
12	.40	6.4	.16	.0640	8.0	.46	.1840	8.8	.36	.1440
13	.20	6.6	.15	.0300	8.8	.36	.0720	9.6	.39	.0780
				1.6948			3.9472			3.9692

1/ See figure 16.9 for definition of Y.

2/ The time to peak of the flood hydrograph is the time of beginning of incremental runoff opposite triangle number 9 plus the time to peak of the unit hydrograph.

contribute to the peak of a composite flood hydrograph at location 1 (0.2 x 13 = 2.6 hours). From table 16.6 note the maximum runoff for one ΔD (0.2 hour) is 0.65 inches, which occurs during the period from 5.8 to 6.0 hours from the beginning of rainfall.

Step 4. Tabulate the runoff in ΔD time increments each way from the maximum ΔD of runoff. There should be at least eight increments of runoff ahead of and four increments of runoff after the maximum increment as shown in table 16.7, column 4, where the ΔD increments of runoff are tabulated opposite the elapsed time after rainfall begins on the watershed.

Step 5. Multiply column 2 by column 4 and tabulate in column 5 of table 16.7.

Step 6. Compute the peak discharge and time to peak of the flood hydrograph at location 1 by multiplying the total of column 5 by the peak discharge of the unit hydrograph.

$$q_p = 1.695 \times 968 = 1640 \text{ cfs}$$

$$T_p = 5.8 + 1 = 6.8 \text{ hours (from beginning of rainfall)}$$

For Location 2:

Step 1. Compute the time increment, ΔD.
From equation 16.12, $\Delta D = .133 \times 6 = .8$ hour

Step 2. Compute q_p the peak discharge for the unit hydrograph. From equation 16.9:

$$q_p = \frac{484 \times 20 \times 1}{\frac{.8}{2} + 3.6} = 2420 \text{ cfs}$$

Step 3. Compute the maximum length of excess rainfall or runoff that adds to the peak of the composite flood hydrograph at location 2 (.8 x 13 = 10.4 hours). From table 16.6 the maximum runoff for one ΔD (.8 hour) is 1.59 inches and occurs during the period from 5.6 to 6.4 hours after the beginning of rainfall.

Step 4. Tabulate the runoff in ΔD time increments each way from the maximum ΔD of runoff. This tabulation is shown in table 16.7, column 7.

Step 5. Multiply column 2 by column 7 and tabulate in column 8.

Step 6. Compute the peak discharge and time to peak as shown in step 6 of example at location 1:

$$q_p = 3.947 \times 2420 = 9550 \text{ cfs}$$

$$T_p = 5.6 + 4.0 = 9.6 \text{ hours (from beginning of rainfall)}$$

Generally, the peak of the composite flood hydrograph can be computed by placing the largest increment of runoff opposite triangle number 9 as shown in table 16.7, column 1 and 4. However, if runoff is irregular, more than one computation may be required before determining the peak of the composite flood hydrograph. Trial 2 also is shown in table 16.7. In this case, the largest increment of runoff is placed opposite triangle number 8. Using the same procedure as in trial 1, the results are:

$$q_p = 3.969 \times 2420 = 9600 \text{ cfs}$$

$$T_p = 6.4 + 4.0 = 10.4 \text{ hours (from beginning of rainfall)}$$

Trial 2 shows that the peak discharge is greater when the largest increment of runoff is placed opposite triangle number 8. Other patterns of runoff may require several computations before the peak discharge is determined.

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SECTION 4

HYDROLOGY

CHAPTER 17. FLOOD ROUTING

by

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CHAPTER 17. FLOOD ROUTING

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SECTION 4

HYDROLOGY

CHAPTER 17. FLOOD ROUTING

Introduction

In the American Society of Civil Engineers' manual, "Nomenclature for Hydraulics," flood routing is variously defined as follows:

<u>routing (hydraulics).</u>--(1) The derivation of an outflow hydrograph of a stream from known values of upstream inflow. The procedure utilizes wave velocity and the storage equation; sometimes both. (2) Computing the flood at a downstream point from the flood inflow at an upstream point, and taking channel storage into account.

<u>routing, flood.</u>-- The process of determining progressively the timing and shape of a flood wave at successive points along a river.

<u>routing, streamflow.</u>--The procedure used to derive a downstream hydrograph from an upstream hydrograph, or tributary hydrographs, and from considerations of local inflow by solving the storage equation.

Routing is also done with mass curves of runoff or with merely peak rates or peak stages of runoff, as well as hydrographs. The routing need not be only downstream because the process can be reversed for upstream routing, which is often done to determine upstream hydrographs from hydrographs gaged downstream. Nor is routing confined to streams and rivers; it is regularly used in obtaining inflow or outflow hydrographs, mass curves, or peak rates in reservoirs, farm ponds, tanks, swamps, and lakes. And low flows are routed, as well as floods. The term "flood routing" covers all of these practices.

The purpose of flood routing in most engineering work is to learn what stages or rates of flow occur, without actually measuring them, at specific locations in streams or structures during passages of floods. The stages or rates are used in evaluating or designing a water-control structure or project. Differences in stages or rates from routings made with and without the structure or project in place show its effects on the flood flows. In evaluations, the differences are translated into monetary terms to show benefits on an easily comparable basis; in design, the differences are used directly in developing or modifying the structure or project characteristics.

The routing process is based on one of the following approaches:

1. Solution of simultaneous partial differential equations of motion and continuity. Simplified versions of the equations are generally used in electronic computer routings; even the simplifications are too laborious for manual routings.
2. Solution of the continuity equation alone. A simplified form of the equation is the basis for many routing methods.
3. Use of inflow-outflow hydrograph relationships.
4. Use of unit hydrograph theory.
5. Use of empirical relationships between inflow and outflow peak stages or rates. Mostly used for large rivers.
6. Use of hydraulic models.

Methods based on the second, third, and fourth approaches are presented in this chapter. The routing operations in the methods can be made numerically by means of an electronic computer, desk calculator, slide rule, nomograph, network chart, or by mental calculations; or graphically by means of an analog machine, special chart, or by successive geometrical drawings. Methods specifically intended for electronic computers or analog machines are neither presented nor discussed.

All methods presented in this chapter are accurate enough for practical work if they are applied as they are meant to be and if data needed for their proper application are used. Advantages and disadvantages of particular methods are mentioned and situations that lead to greater or lesser accuracy of a method are pointed out, but there is no presentation of tests for accuracy or of comparisons between routed and gaged hydrographs.

SCS electronic computer program

The electronic computer program now being used in SCS watershed evaluations contains two methods of flood routing. The Storage-Indication method is used for routing through reservoirs and the Convex method for routing through stream channels. Manual versions of both methods are described in this chapter.

References

Each of the following references contains general material on flood routing and descriptions of two or more methods. References whose main subject is not flood routing but which contain a useful example of routing are cited in the chapter as necessary.

1. Thomas, H. A., 1937, The hydraulics of flood movements in rivers: Pittsburg, Carnegie Inst. Tech., Eng. Bull. Out of print but it can be found in most libraries having collections of engineering literature.

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5. Yevdjevich, Vujica M., 1964, Bibliography and discussion of flood-routing methods and unsteady flow in channels: U.S. Geol. Survey Water-Supply Paper 1690. Prepared in cooperation with the Soil Conservation Service.

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Summary of chapter contents

The remainder of this chapter is divided into four parts: elevation-storage and elevation-discharge relationships, reservoir routing methods, channel routing methods, and unit-hydrograph routing methods. In the first part, some relationships used in reservoir or channel routing are discussed and exhibits of typical results are given; in the second, the continuity equation is discussed and methods of using it in reservoir routings are shown in examples of typical applications; in the third, the theory of the Convex method is presented and examples of typical applications in channel routings are given; and in the fourth, the unit hydrograph theory is discussed and methods of applying it in systems analysis are shown in examples using systems of floodwater-retarding structures.

Elevation-Storage and Elevation-Discharge Relationships

In the examples of routing through reservoirs and stream channels it will be necessary to use elevation-storage or elevation-discharge curves (or both) in making a routing or as a preliminary to routing. Preparation of such curves is not emphasized in the examples because their construction is described in other SCS publications. The relationships are briefly discussed here as preliminary material; exhibits of tables and curves used in routings are given here and in some of the examples. Conversion equations used in preparing the tables and curves are given in Table 17-1.

Elevation storage relationships for reservoirs

Table 17-2 is a working table that shows data and computed results for an elevation-storage relationship to be used in some of the examples given later. Columns 1 and 7 or 1 and 8 give the relationship in different units of storage.

The relationship is developed from a contour map (or equivalent) of the reservoir area and the table is a record of the computations that were made. Once the map is available, the work goes as follows: (1) select contours close enough to define the topography with reasonable accuracy and tabulate the contour elevations in column 1; (2) determine the

reservoir surface area at each elevation; for this table the areas were determined in square feet as shown in column 2 and converted to acres as shown in column 3; (3) compute average surface areas as shown in column 4; (4) tabulate the increments of depth in column 5; (5) compute the increments of storage for column 6 by multiplying an average area in column 4 by its appropriate depth increment in column 5; (6) accumulate the storage increments of column 6 to get accumulated storage in column 7 for each elevation of column 1; (7) convert storages of column 7 to storages in another unit, if required, and show them in the next column. The relationship of data in columns 1 and 8 is plotted in figure 17.1 as an elevation-storage curve.

Table 17-1. Equations for conversions of units

Conversion	Equation No.
cfs-hours = 12.1 (AF)	(Eq. 17.1)
cfs-days = 0.504 (AF)	(Eq. 17-2)
inches = (AF)/53.3 A	(Eq. 17-3)
$q_{id} = q_{cfs}/26.9\ A$	(Eq. 17-4)
$q_{ih} = q_{cfs}/645\ A$	(Eq. 17-5)
$q_{ad} = 1.98\ q_{cfs}$	(Eq. 17-6)
$q_{ah} = 0.0821\ q_{cfs}$	(Eq. 17-7)
$S_x = L\ (A_x)/3600$	(Eq. 17-8)
$S'_x = L\ (A_x)/297$	(Eq. 17-9)

where A = drainage area in square miles

A_x = cross section end-area in square feet for discharge x

AF = acre-feet

L = reach length in feet

q_{ad} = discharge in acre-feet per day

q_{ah} = discharge in acre-feet per hour

q_{cfs} = discharge in cfs

q_{id} = discharge in inches per day

q_{ih} = discharge in inches per hour

S_x = reach storage in cfs-hours for a given discharge x

S'_x = reach storage in acre-feet for a given discharge x

Table 17-2. Elevation-storage relationship for a reservoir.

Elevation (feet)	Surface area (sq.ft.)	Surface area (acres)	Average surface area (acres)	Δ depth (feet)	Δ storage (AF)	Storage (AF)	Storage (inches)
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
570	0	0				0	0
			4.82	2.00	9.64		
572	420,000	9.64				9.64	.022
			18.36	2.00	36.72		
574	1,180,000	27.09				46.36	.109
			38.34	2.00	76.68		
576	2,374,000	54.50				123.04	.288
			71.62	4.00	286.49		
580	3,866,000	88.75				409.53	.960
			106.67	5.00	533.35		
585	5,427,000	124.59				942.88	2.210
			153.60	5.00	768.00		
590	7,954,000	182.60				1710.88	4.010
			205.64	5.00	1028.20		
595	9,961,000	228.67				2739.08	6.420
			250.01	5.00	1250.05		
600	11,820,000	271.35				3989.13	9.351

Elevation-discharge relationships for reservoirs

The elevation-discharge relationship for a reservoir is made using elevations of the reservoir and discharges of the spillways to be used in a routing. A typical relationship for a 2-stage principal spillway is given by columns 1 and 6 of Table 17-3 for discharges in cfs, and in columns 1 and 7 for discharges in in./day. The procedure for developing the relationship will not be given here because sufficient charts, equations, and examples for principal spillways are given in NEH-5 and in ES-150 through 153, and for emergency spillways in ES-98 and ES-124. Table 17-3 illustrates a useful way of keeping the work in order: by tabulating the data for different types of flow in separate columns, and by keeping the two stages separate, the total discharges are more easily summed. Note that the totals in cfs are not merely sums of all cfs in a row; the operation of the spillway must be understood when selecting the discharges to be included in the sum. To combine the principal spillway flow with emergency spillway flow a column for the emergency spillway discharges is added between columns 5 and 6, and totals in column 6 must include those discharges where appropriate. Column 7 gives discharges converted from those in column 6; it is shown because this table is used in examples given later and that particular unit of flow is required (see Figure 17-1).

Storage-discharge relationships for reservoirs

If the elevation-storage and elevation-discharge relationships are to be used for many routings it is more convenient to use them as a storage-discharge relationship. The relationships are combined by plotting a graph of storage and elevation, another of discharge and elevation, and, while referring to the first two graphs, making a third by plotting storage for a selected elevation against discharge for that elevation; for a typical curve see Figure 17-2. The storage-discharge curve can also be modified for ease of operations with a particular routing method; for a typical modification see Figure 17-6 and step 4 of Example 17-4.

Elevation, stage, storage, discharge relationships for streams

It is common practice to divide a stream channel into reaches (see Chapter 6) and to develop storage or discharge relationships for individual reaches rather than the stream as a whole. A stream elevation- or stage-discharge curve is for a particular cross section. If a reach has several cross sections within it they are all used in developing the working tools for routing. Some routing methods require the use of separate discharge curves for the head and foot of a reach; such methods are not presented in this chapter.

Elevation- or stage-discharge curves for cross sections or reaches are prepared as shown in Chapter 14. They will not be discussed here.

Elevation- or stage-storage curves for a reach can be prepared using the procedure for reservoirs but ordinarily a modified approach is used and the storage-discharge curve prepared directly. Table 17-4 is a working table for developing such a curve. The work is based on the assumption that steady flow occurs in the reach at all stages of flow. The reach used in Table 17-4 has four cross sections so that a weighting method

Table 17-3 Elevation-discharge relationship for a 2-stage principal spillway.

Elevation (feet)	Discharge: First stage: Weir (cfs)	Orifice (cfs)	Second stage: Weir (cfs)	Pipe (cfs)	Total (cfs)	Total (in./day)
(1)	(2)	(3)	(4)	(5)	(6)	(7)
580.2	0				0	0
580.7	4.1				4.1	.019
581.2	11.6				11.6	.054
581.7	21.3				21.3	.099
582.2	32.8				32.8	.153
582.7	45.8				45.8	.213
583.2	60.3	0			60.3	.281
583.7	75.3	89.5			75.3	.350
584.2	92.8	101			92.8	.432
585.2	130	120			120	.559
586.0	162	133			133	.620
587.0	206	149	0	0	149	.694
587.5		159	44.6	343	204	.950
588.0		163	126	347	289	1.346
588.5		170	232	353	353	1.644
589.0		176	357	357	357	1.663
589.5		182	499	361	361	1.680
590.0			656	365	365	1.697
590.2			722	367	367	1.707
591.0				374	374	1.740
592.0				382	382	1.778
595.0				401	401	1.863
600.0				432	432	2.003

is needed; with only one or two sections the weighting is eliminated but the reach storage is less well defined. Development of the storage-discharge curve goes as follows: (1) select a series of discharges from zero to a discharge greater than any to be routed and tabulate them in column 1; (2) enter the stage-discharge curve for each cross section with a discharge from column 1 and find the stage; (3) enter the stage-end-area curve for that section with the stage from step 2 and find the area at that stage, tabulating areas for all sections as shown in columns 2, 3, 4, and 5; (4) determine the distances between cross sections and compute the weights as follows:

From cross section	To cross section	Distance (feet)	Weight
1	2	1000	0.10
2	3	6000	.60
3	4	3000	.30
	Sum:	10000	

with the weight for sub-reach 1-2 being 1000/10000 = 0.10, and so on; (5) compute weighted end areas for columns 6, 7, and 8; for example, at a discharge of 3,500 cfs cross section 1 has an end area of 2,500 square feet and section 2 has 640 square feet, and the weighted end area is 0.10(2500 + 640)/2 = 157 square feet; (6) sum the weighted areas of columns 6, 7, and 8 for each discharge, tabulating the sums in column 9; (7) compute storages in column 10 by use of Equation 17-8 or 17-9, whichever is required; for example, at a discharge of 3,500 cfs the storage in cfs-hrs is S_{3500} = 10000(1189)/3600 = 3300 cfs-hrs, by a slide-rule computation. The storage-discharge curve is plotted using data from columns 1 and 10. Data of those columns can be used in preparing the working curve for routing. How this is done depends on the routing method to be used. For the Storage-Indication method the working curve is prepared as shown in Example 17-4.

Table 17-4 Working table for a storage-discharge relationship

Out-flow	Cross section end-areas				Weighted end-areas			Avg. end-areas	Stor-age
	1	2	3	4	1-2	2-3	3-4		
(cfs)	(sq.ft)	(sq.ft)	(sq.ft)	(sq.ft)	(sq.ft)	(sq.ft)	(sq.ft)	(sq.ft)	(cfs-hrs)
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)
0	0	0	0	0	0	0	0	0	0
50	40	27	21	33	3	14	8	25	70
150	90	44	44	64	7	26	16	49	164
300	150	83	83	100	12	50	27	89	248
800	470	180	220	325	32	120	82	234	651
1500	950	310	460	700	63	231	174	468	1302
3500	2500	640	1200	2000	157	552	480	1189	3300
5000	3250	860	1700	2700	205	768	660	1633	4540
7000	4400	1050	2050	3400	272	930	819	2021	5620
10000	5800	1300	2550	4500	355	1155	1055	2565	7130

Reservoir Routing Methods

Reservoirs have the characteristic that their storage is closely related to their outflow rate. In reservoir routing methods the storage-discharge relation is used for repeatedly solving the continuity equation, each solution being a step in delineating the outflow hydrograph. A reservoir method is suited for channel routings if the channel has the reservoir characteristic. Suitable channels are those with swamps or other flat areas in the routing reach and with a constriction or similar control at the foot of the reach. There is an exception to this: a reservoir method is also suitable for routing through any stream reach if the inflow hydrograph rises and falls so slowly that nearly steady flow occurs and makes storage in the reach closely related to the outflow rate. Examples in this part show the use of reservoir methods for both reservoirs and stream channels.

The Continuity Equation

The continuity equation used in reservoir routing methods is concerned with conservation of mass: For a given time interval, the volume of inflow minus the volume of outflow equals the change in volume of storage. The equation is often written in the simple form:

$$\Delta t\,(\bar{I} - \bar{O}) = \Delta S \qquad \text{(Eq. 17-10)}$$

where Δt = a time interval

$\bar{I}$ = average rate of inflow during the time interval

$\bar{O}$ = average rate of outflow during the time interval

ΔS = change in volume of storage during the time interval

In most applications of the continuity equation the flow and storage variables are expanded as follows:

$$\bar{I} = \frac{I_1 + I_2}{2}\ ; \qquad \bar{O} = \frac{O_1 + O_2}{2}\ ; \qquad \Delta S = S_2 - S_1$$

so that Equation 17-10 becomes:

$$\frac{\Delta t}{2}(I_1 + I_2) - \frac{\Delta t}{2}(O_1 + O_2) = S_2 - S_1 \qquad \text{(Eq. 17-11)}$$

where $\Delta t = t_2 - t_1$ = time interval; t_1 is the time at the beginning of the interval and t_2 the time at the end of the interval

I_1 = inflow rate at t_1

I_2 = inflow rate at t_2

O_1 = outflow rate at t_1

O_2 = outflow rate at t_2

S_1 = storage volume at t_1

S_2 = storage volume at t_2

When routing with Equation 17-10 the usual objective is to find $\overline{O}$, with Equation 17-11 find O_2; this means that the equations must be rearranged in some more convenient working form. It is also necessary to use the relationship of outflow to storage in making a solution. Most reservoir routing methods now in use differ only in their arrangement of the routing equation and in their form of the storage-outflow relationship.

It is necessary to use consistent units with any routing equation. Some commonly used sets of units are:

Time	Rates		Volumes		
	Inflow	Outflow	Inflow	Outflow	Storage
Hours	cfs	cfs	cfs-hrs	cfs-hrs	cfs-hrs
days	cfs	cfs	cfs-days	cfs-days	cfs-days
days	AF/day	AF/day	AF	AF	AF
hours	in./hr	in./hr	inches	inches	inches
days	in./day	in./day	inches	inches	inches

Methods and Examples

Two methods of reservoir routing based on the continuity equation are presented in this section, a mass-curve method and the Storage-Indication method. The mass-curve method is given because it is one of the most versatile of all reservoir methods. It can be applied numerically or graphically; examples of both versions are given. The Storage-Indication method is given because it is the method used at the present time in the SCS electronic computer program for watershed evaluations and because it is a widely used method for both reservoir and channel routings. Examples of reservoir and channel routing are given.

Mass-Curve Method: Numerical Version - According to item 52 in reference 5, a mass-curve method of routing through reservoirs was already in use in 1883. Many other mass-curve methods have since been developed. The method described here is similar to a method given in King's "Handbook of Hydraulics," 3rd edition, 1939, pages 522-527; another resembling it is given in "Design of Small Dams," U. S. Bureau of Reclamation, 1960, pages 250-252.

The method requires the use of elevation-storage and elevation-discharge relationships either separately or in combination. The input is the mass (or accumulated) inflow; the output is the mass outflow, outflow hydrograph, and reservoir storage. The routing operation is a trial-and-error process when performed numerically, but it is simple and easily done. Each operation is a solution of Equation 17-10 rewritten in the form:

$$MI_2 - (MO_1 + \overline{O}\,\Delta t) = S_2 \qquad \text{(Eq. 17-12)}$$

where MI_2 = mass inflow at time 2

MO_1 = mass outflow at time 1

$\overline{O}$ = average discharge during the routing interval

Δt = routing interval = time 2 minus time 1

S_2 = storage at time 2

The routing interval can be either variable or constant. Usually it is more convenient to use a variable interval, making it small for a large change in mass inflow and large for a small change. The PSMC of Chapter 21 are tabulated in intervals especially suited for this method of routing.

The following example shows the application of the method in determining minimum required storage for a floodwater-retarding structure by use of a PSMC from Chapter 21.

Example 17-1.--Determine the minimum required storage, by SCS criteria, for a floodwater-retarding structure having the drainage area use in Example 21-2 of Chapter 21. Use the data and results of that example for this structure. Work with volumes in inches and rates in inches per day; round off all results to the nearest 0.01 inch.

1. Develop an elevation-discharge curve for the structure. A curve for the principal spillway discharges is needed for this routing. Columns 1 and 7 of Table 17-3 will be used for this structure. The elevation-discharge curve is plotted in Figure 17-1.

2. Develop an elevation-storage curve for the structure. Columns 1 and 8 of Table 17-2 will be used for this structure. The elevation-storage curve is plotted in Figure 17-1.

(Note: The curves of steps 1 and 2 can be combined into a storage-discharge curve as shown by the inset of Figure 17-2. This curve is a time-saver if more than one routing is made.)

3. Develop and plot the curve of mass inflow (PSMC). The PSMC developed in Example 21-2, and given by columns 1 and 7 of Table 21-7, will be used for this example. The plotted mass inflow is shown in Figure 17-2. The plotting is used as a guide in the routing and later used to show the results but it is not essential to the method.

4. Prepare an operations table for the routing. Suitable headings and arrangement for an operations table are shown in Table 17-5.

5. Determine the reservoir storage for the start of the routing.
If the routing is to begin with some storage already occupied then either the amount in storage is entered in the first line or column 5 of the operations table (as done in Example 17-2) or the elevation-storage curve is modified to give a zero storage for the first line. In this example the sediment or dead storage, which is not to be used in the routing, occupies the reservoir to elevation 580.2 feet as shown in Figure 17-1. Storage at that elevation is 1.00 inches and because this is a whole scale unit the storage curve for routing is easily obtained by shifting the point of origin as shown in Figure 17-1. Ordinarily, if the Sediment or dead storage is some fractional quantity it is better to re-plot the curve to show zero storage at the elevation where the routing begins.

6. Determine the spillway discharge at the start of the routing.
If the spillway is flowing at the start of the routing the discharge rate is entered in the first line of column 7 of Table 17-5 (see Example 17-2). For this example the starting rate is zero.

7. Do the routing.
The trial-and-error procedure goes as follows:

a. Select a time and tabulate it in column 1, Table 17-5. For this example the times used will be those given for the PSMC in Table 21-7, except for occasional omissions unimportant for this routing.

b. Compute Δt and enter the result in column 2.

c. Tabulate in column 3 the mass inflow for the time in column 1. The entries for this example come from column 7 of Table 21-7.

d. Assume a mass outflow amount and enter it in column 4.

e. Compute the reservoir storage, which is the inflow of column 3 minus the outflow of column 4, and enter it in column 5.

f. Determine the instantaneous discharge rate of the spillway. Using the elevation-storage curve of Figure 17-1, find the elevation for the storage of column 5; with that elevation enter the elevation-discharge curve and find the discharge, tabulating it in column 6. If a storage-discharge curve is being used, simply enter the curve with the storage and find the corresponding discharge.

g. Compute the average discharge for Δt. The average is always the arithmetic mean of the rate determined in step f and the rate for the previous time. For the time 0.5 days the rate in column 6 is 0.03 in./day; for the previous time the rate is zero; the average rate is $(0 + 0.03)/2 = 0.015$, which

Table 17-5. Operations table for the mass-curve method of routing for Example 17-1.

Time Acc. (days)	Time Δt (days)	Acc. inflow (in.)	Assumed acc. outflow (in.)	Res. volume (in.)	Spillway discharge Inst. (in./day)	Spillway discharge Avg. (in./day)	Outflow for Δt (in.)	Acc. outflow (in.)
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)
0		0	0	0	0		0	0
.5	0.5	.13	.01	.12	.03	0.02	.01	.01
1.0	.5	.30	.04	.26	.08	.06	.03	.04
2.0	1.0	.69	.15	.54	.21	.14	.14	.18
			.17	.52	.19	.14	.14	.18
3.0	1.0	1.14	.40	.74	.31	.25	.25	.43
			.42	.72	.29	.24	.24	.42
3.5	.5	1.42	.60	.82	.36	.32	.16	.58
			.59	.83	.37	.33	.16	.58
4.0	.5	1.76	.75	1.01	.46	.42	.21	.79
			.78	.98	.44	.40	.20	.78
4.4	.4	2.11	1.02	1.09	.52	.48	.19	.97
			.98	1.13	.53	.48	.19	.97
4.8	.4	2.62	1.20	1.42	.61	.57	.23	1.20
5.0	.2	3.38	1.35	2.03	1.00	.80	.16	1.36
5.1	.1	4.07	1.45	2.62	1.66	1.33	.13	1.49
			1.48	2.59	1.65	1.32	.13	1.49
5.2	.1	4.43	1.70	2.73	1.68	1.66	.17	1.66
			1.67	2.76	1.68	1.66	.17	1.66
5.3	.1	4.66	1.85	2.81	1.69	1.68	.17	1.83
			1.84	2.82	1.69	1.68	.17	1.83
5.4	.1	4.81	2.10	2.71	1.67	1.68	.17	2.00
			2.01	2.80	1.68	1.68	.17	2.00
5.6	.2	5.05	2.30	2.75	1.67	1.68	.34	2.34
			2.33	2.72	1.67	1.68	.34	2.34
6.0	.4	5.38	2.80	2.58	1.66	1.66	.66	3.00
			2.95	2.43	1.64	1.66	.66	3.00
			3.00	2.38	1.60	1.64	.66	3.00
6.5	.5	5.70	3.80	1.90	.80	1.20	.60	3.60
			3.70	2.00	.94	1.27	.64	3.64
			3.65	2.05	1.04	1.32	.66	3.66
7.0	.5	5.98	4.10	1.88	.70	.89	.44	4.10
8.0	1.0	6.43	4.80	1.63	.66	.68	.68	4.79
etc.	etc.	etc.	etc.	etc.	etc.	etc.	etc.	etc.

Mass outflow is plotted using entries in column 4 or column 9. The outflow hydrograph is plotted using column 6, which gives instantaneous rates at the accumulated times shown in column 1.

is rounded to 0.02 in./day. For the time 1.0 days the average is (0.03 + 0.08)/2 = 0.055, which is rounded to 0.06 in./day; and so on.

h. Compute the outflow for Δt. Multiply the Δt of column 2 by the average rate of column 7 and get the increment of outflow for column 8.

i. Add the outflow increment of column 8 to the total of column 9 for the previous time and tabulate the sum in column 9.

j. Compare the mass outflow of column 9 with the assumed mass outflow of column 4. If the two entries agree within the specified degree of accuracy (0.01 inch, in this routing) then this routing operation is complete and a new one is begun with step a. If the two entries do not agree well enough then assume another mass outflow for column 4 and repeat steps e through j.

8. Determine the minimum required storage.
Examine the entries in column 5 and find the largest entry, which is 2.82 inches at 5.3 days. This is the minimum required storage.

The routing gives the reservoir storages in column 5, outflow hydrograph in column 6, and mass outflow in column 9, for the times of column 1. Unless the results are to be used in a report or exhibit, the routing is usually carried only far enough past the time of maximum storage to ensure that no larger storage will occur. The mass inflow and outflow for this example are plotted in Figure 17-2, with outflow shown only to 8.0 days. If the mass outflow plotting is made during the routing the trend of the curve indicates the best assumption for the next step in column 4.

The next example shows how the routing proceeds when it must start with the reservoir containing live storage and the spillway discharging.

Example 17-2.--For the same reservoir used in Example 17-1, determine the elevation and amount of storage remaining in the reservoir after 10 days of drawdown from the minimum level allowed by SCS criteria. The base flow used in developing the PSMC (see Example 21-2) is assumed to continue at the same rate throughout the routing. Round all work to the nearest 0.01 inch.

1. Determine the storage volume in the reservoir and the spillway discharge for the start of the routing.
SCS criteria permit the drawdown routing to start with storage at the maximum elevation attained in the routing of the PSH or PSMC used in determining the minimum required storage, even though the structure may be designed to contain more than the minimum storage. For this example the starting storage of 2.82 inches is found in column 5, and the associated discharge of 1.69 in./day in column 6, of Table 17-5 in the line for 5.3 days.

2. Prepare an operations table for the routing.
Ordinarily the suitable headings and arrangement are those of Table 17-5, but if base flow, snowmelt, or upstream releases must be included (base flow in this routing) then one or more additional columns are needed. Table 17-6 shows headings and arrangement suitable for this example.

3. Do the routing.
The procedure of step 7, Example 17-1, is slightly modified for this routing. The first line of data in the operations table must contain the initial reservoir volume in column 4 and the initial spillway discharge in column 7. Accumulated base flow is added to the initial value of column 4 to give the "accumulated inflow" of that column. In all other respects the routing procedure is that of step 7, Example 17-1.

4. Determine the storage remaining after 10 days of drawdown.
The entry in column 6 at day 10 shows that the remaining storage is 0.20 inches, which is at elevation 581.1 feet.

The routing for this example has been carried to 14 days to show that when the inflow rate is steady, as it is in this case (0.045 in./day), then the outflow rate eventually also becomes steady at the same rate. The larger the steady rate of inflow the sooner the outflow becomes steady. Note that if the routing had been done with an accuracy to the nearest 0.001 inch, the outflow rate would be 0.045 in./day, the base flow rate.

The mass inflow, storage, and mass outflow curves for this example are shown in Figure 17-3. Note that the work is accurate to the nearest 0.01 inch, therefore the curves must follow the plotted points within that limit. Slight irregularities in the smooth curves are due to slope changes in the storage-discharge curve.

Mass-Curve Method: Direct Version.- It is easy enough to eliminate the trial-and-error process of the mass-curve method but the resulting "direct version" is much more laborious than the trial-and-error version. To get a direct version the working equation is obtained from Equation 17-12 as follows.

The average discharge $\overline{O}$ in Equation 17-12 is $(O_1 + O_2)/2$ so that the equation can be written:

$$MI_2 - MO_1 - \frac{\Delta t}{2}(O_1 + O_2) = S_2 \qquad \text{(Eq. 17-13)}$$

Because O_2 as well as S_2 is unknown it is necessary to make combinations of S and O to get direct solutions in the routing operation. At any time, mass outflow is equal to mass inflow minus storage, or:

$$MO_1 = MI_1 - S_1 \qquad \text{(Eq. 17-14)}$$

Table 17-6 Operations table for determining storage after 10 days of drawdown for Example 17-2.

Time Acc. (days)	Time Δt (days)	Acc. base flow* (in.)	Acc. in-flow (in.)	As-sumed acc. outflow (in.)	Res. vol-ume (in.)	Spillway discharge Inst. (in./day)	Spillway discharge Avg. (in./day)	Out-flow for Δt (in.)	Acc. out-flow (in.)
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)
0		0	2.82		2.82	1.69			0
.2	0.2	.01	2.83	0.34	2.49	1.66	1.67	0.33	.33
.4	.2	.02	2.84	.64	2.20	1.35	1.50	.30	.63
.6	.2	.03	2.85	.92	1.93	.87	1.11	.22	.85
				.86	1.99	.98	1.16	.23	.86
1.0	.4	.04	2.86	1.20	1.66	.66	.82	.33	1.19
1.5	.5	.07	2.89	1.50	1.39	.60	.63	.32	1.51
2.0	.5	.09	2.91	1.80	1.11	.53	.56	.28	1.79
2.5	.5	.11	2.93	2.03	.90	.37	.45	.22	2.01
				2.01	.92	.38	.46	.23	2.02
3.0	.5	.14	2.96	2.23	.73	.27	.32	.16	2.18
				2.19	.77	.29	.34	.17	2.19
3.5	.5	.16	2.98	3.30	.68	.24	.26	.13	2.32
				2.32	.66	.23	.26	.13	2.32
4.0	.5	.18	3.00	2.42	.58	.20	.22	.11	2.43
4.5	.5	.20	3.02	2.52	.50	.17	.18	.09	2.52
5.0	.5	.22	3.04	2.59	.45	.15	.16	.08	2.60
6.0	1.0	.27	3.09	2.73	.36	.12	.14	.14	2.74
7.0	1.0	.32	3.14	2.85	.29	.09	.10	.10	2.84
8.0	1.0	.36	3.18	2.94	.24	.07	.08	.08	2.92
				2.93	.25	.08	.08	.08	2.92
9.0	1.0	.40	3.22	3.00	.22	.07	.08	.08	3.00
10.0	1.0	.45	3.27	3.07	.20	.07	.07	.07	3.07
11.0	1.0	.50	3.32	3.13	.19	.06	.06	.06	3.13
12.0	1.0	.54	3.36	3.19	.17	.05	.06	.06	3.19
13.0	1.0	.58	3.40	3.25	.15	.04	.04	.04	3.23
				3.24	.16	.05	.05	.05	3.24
14.0	1.0	.63	3.45	3.29	.16	.05	.05	.05	3.29
etc.	etc.	etc.	etc.	etc.	etc.	etc.	etc.	etc.	etc.

* At a rate of 0.045 inches per day.

Substituting $MI_1 - S_1$ for MO_1 in Equation 17-13 and rearranging gives:

$$MI_2 - MI_1 + (S_1 - \frac{\Delta t}{2} O_1) = S_2 + \frac{\Delta t}{2} O_2 \qquad \text{(Eq. 17-15)}$$

which is the working equation for the direct version. Working curves of O_1 and $(S_1 - (\Delta t\ O_1)/2)$ and of O_2 and $(S_2 + (\Delta t\ O_2)/2)$ are needed for routing.

Other arrangements of working equations can also be obtained from Equation 17-12. Equation 17-15 is the mass-curve version of the Storage-Indication method, which is described later in this part. Routing by use of Equation 17-15 takes about twice as much work as routing by the Storage-Indication method.

Examples of direct versions of the mass-curve method are not given in this chapter because the trial-and-error version is more efficient in every respect.

Mass-Curve Method: Graphical Version.- The graphical version of the mass-curve method is in a sense a direct version because there is no trial-and-error involved. The graphical version is usually faster than the trial-and-error version if the routing job is simple. For complex jobs the trial-and-error version is more efficient and its results more easily reviewed. For any routing it gives mass outflow, storage, and the outflow hydrograph; the graphical version gives only the mass outflow and storage. The following example shows the use of the graphical version with the data and problem of Example 17-1.

Example 17-3.--Use the graphical version of the mass-curve method to determine the minimum required storage for the structure used in Example 17-1. Use the data of that example.

1. Develop an elevation-discharge curve for the structure.
The curve used in Example 17-1 will be used here.

2. Develop an elevation-storage curve for the structure.
The curve used in Example 17-1 will be used here.

3. Prepare a working table for the routing.
Using the curves of steps 1 and 2, select enough discharges on the discharge curve to define the curve accurately and tabulate them in column 2, Table 17-7. Tabulate the associated elevations in column 1 and storages at those elevations in column 4. Compute average discharges from column 2 for column 3. The designations in column 5 show which line is associated with each pair of storages shown on Figure 17-4. Thus, line A applies when the storage is between 0 and 0.18 inches; line B when it is between 0.18 and 0.40 inches; and so on.

4. Plot the mass inflow.
The PSMC used in Example 17-1 is used here. It is plotted in Figure 17-4.

5. Do the routing.
The work is done on the graph of mass inflow, Figure 17-4. Table 17-7 is used during the work. The procedure goes as follows:

a. Draw line A with its origin at the beginning of mass inflow and with its slope equal to the associated average discharge (column 3 of Table 17-7), which is 0.025 in./day. This is the first portion of the mass outflow curve.

(Note: Every part of the line of mass outflow must fall on or below the mass inflow curve. If some part is above the inflow, determine the slope and storage limits for a line with a flatter slope and use it instead.)

b. Determine the time at which the difference between mass inflow and line A is equal to the larger of the storage limits for line A, in this case 0.18 inches, which occurs at 0.65 days. This is the point of origin for line B.

c. Draw line B with its origin at the point found in step b and with a slope of 0.09 in./day.

d. Determine the time at which the difference between mass inflow and line B is equal to the larger of the storage limits for line B, in this case 0.40 inches, which occurs at 1.50 days. This is the point of origin for line C.

e. Repeat the procedure of steps c and d with lines C, D, E, etc., until the storage being used is so large it exceeds the possible difference between mass inflow and mass outflow. For this example this occurs with line H. The parallel line above it shows that the associated storage of 3.44 inches falls above the mass inflow line. When this step is reached the required storage is obtained by taking the maximum difference between line H and the mass inflow curve. The difference occurs at the point on the mass inflow curve where a line parallel to line H is tangent to the inflow curve. For this example it is 2.80 inches at 5.33 days. This step completes the routing.

The graphical method can also be used for routings starting with some storage occupied and with the spillway discharging. For the problem used in Example 17-2 the graphical method starts with line H and continues with lines G, F, E, D, C, B, and A in that order. The results are shown in Figure 17-5. The storage after 10 days of drawdown is 0.18 inches, which is nearly the same as found in Example 17-2. Differences between results of the two methods are due mainly to the use of small-scale graphs for working curves; larger scales increase the accuracy. Note that line A in Figure 17-5 is flatter than the line of accumulated base flow. This indicates that the flow becomes steady at or near 10 days and that the dashed line (parallel to mass inflow) is the actual outflow.

Table 17-7 Working table for the graphical version of the mass-curve method for Example 17-3.

Elevation (feet)	Spillway discharge Inst. (in./day)	Spillway discharge Avg. (in./day)	Storage (inches)	Designation on Fig. 17-4
(1)	(2)	(3)	(4)	(5)
580.2	0		0	
		0.025		line A
581.0	.05		.18	
		.09		line B
582.0	.13		.40	
		.23		line C
583.5	.33		.80	
		.42		line D
584.6	.52		1.09	
		.61		line E
587.0	.70		1.86	
		.95		line F
587.8	1.20		2.16	
		1.42		line G
588.4	1.64		2.38	
		1.69		line H
591.0	1.74		3.44	
		1.77		line I
592.5	1.80		4.15	

Storage-Indication Method.- Reservoir routing methods that are also used for stream routings are generally discharge, not mass, methods because it is usually only the discharge hydrograph that is wanted. The Storage-Indication method, which has been widely used for channel and reservoir routings, has discharge rates as input and output. The method was given in the 1955 edition of NEH-4, Supplement A. Example 17-4, below is the same example used in that publication except for minor changes.

The Storage-Indication method uses Equation 17-11 in the form:

$$\overline{I} + \frac{S_1}{\Delta t} - \frac{O_1}{2} = \frac{S_2}{\Delta t} + \frac{O_2}{2} \qquad \text{(Eq. 17-16)}$$

where $\overline{I} = (I_1 + I_2)/2$. The values of $\overline{I}$ are either taken from midpoints of routing intervals of plotted inflow hydrographs or computed from inflows tabulated at regular intervals. A working curve of O_2 plotted against $(S_2/\Delta t) + (O_2/2)$ is necessary for solving the equation.

In channel routing the Storage-Indication method has the defect that outflow begins at the same time inflow begins so that presumably the inflow at the head of the reach passes instantaneously through the reach regardless of its length. This defect is not serious if the ratio T_t/T_p is about 1/2 or less, where T_p is the inflow hydrograph time to peak and T_t is a travel time defined as:

$$T_t = \frac{L\ A}{3600\ q} = \frac{L}{3600\ V} \qquad \text{(Eq. 17-17)}$$

where T_t = reach travel time in hours; the time it takes a selected steady-flow discharge to pass through the reach

L = reach length in feet

A = average end-area for discharge q in square feet

q = selected steady-flow discharge is cfs

V = q/A = average velocity of discharge q in fps

In determining T_t the discharge q is usually the bank-full discharge under steady flow conditions (see Chapter 15).

Another defect of the Storage-Indication method, for both channel and reservoir routing, is that there is no rule for selecting the proper size of routing interval. Trial routings show that negative outflows will occur during recession periods of outflow whenever Δt is greater than $2\ S_2/O_2$ (or whenever $O_2/2$ is greater than $S_2/\Delta t$). This also means that rising portions of hydrographs are being distorted. In practice, to avoid these possibilities, the working curve can be plotted as shown in Figure 17-6; if any part of the working curve falls above the line of equal values then the entire curve should be discarded and a new one made using a smaller value of Δt. For channel routing the possibility of negative outflows is usually excluded by taking Δt less than T_t.

The following example shows the use of the Storage-Indication method in channel routing. The example is the one used in the 1955 edition of NEH-4, Supplement A, with some minor changes.

Example 17-4.--Use the Storage-Indication method of reservoir routing to route the inflow hydrograph of Figure 17-7 through the stream reach of Table 17-4.

1. <u>Prepare the storage-discharge relationship for the reach</u>.
This is done in Table 17-4 and the text accompanying it.

2. <u>Determine the reach travel time</u>.
This is done using Equation 17-17. Table 17-4 and the accompanying text supply the following data: L = 10,000 feet and for a bank-full discharge of 800 cfs as q the end-area A = 234 square feet. Then by Equation 17-17, T_t = 10000(234)/3600(800) = 0.813 hours.

3. <u>Select the routing interval</u>.
The routing interval for this example will be 0.5 hours, which is less than the travel time of step 2 and which is a convenient size for the given inflow hydrograph. (See the discussion in the text accompanying Equation 17-30 for further information on the selection of reach routing intervals.)

4. <u>Prepare the working curve</u>.
Use the storage-discharge relationship of step 1, which is given in columns 1 and 10 of Table 17-4. These two columns are reproduced as columns 1 and 3 of Table 17-8, the working table; columns 2, 4, and 5 of the table are self-explanatory. The working curve is plotted using columns 1 and 5. The finished curve is shown in Figure 17-6.

5. <u>Prepare the operations table</u>.
Suitable headings and arrangement for an operations table are shown in Table 17-9.

6. <u>Enter times and inflows in the operations table</u>.
Accumulated time in steps of the routing interval is shown in column 1 of Table 17-9. $\bar{I}$ values read from midintervals on the inflow hydrograph of Figure 17-7 are shown in column 2.

7. <u>Do the routing</u>.
The procedure is shown in Table 17-10. The routing results are shown in columns 3 and 4 of Table 17-9. The outflow hydrograph given in column 4 is plotted in Figure 17-7.

In routing through channels it is generally necessary to add local inflow to the routed outflow. The method of doing this is described later in the part on channel routing methods.

The Storage-Indication procedure for reservoir routing is identical with that for channel routing except that there is no need to determine a travel time. The following example shows the reservoir procedure. The problem and data of Example 17-1 are used in order to allow a comparison of procedures and results.

Example 17-5.--Use the Storage-Indication method to determine the minimum required storage for the structure used in Example 17-1. Use the data of that example where applicable. Make the routing with discharges in cfs.

1. Develop an elevation-discharge curve for the structure.
The curve used in Example 17-1 will be used here. That curve is for discharges in in./hr. Ordinarily when cfs are to be used the curve is developed in that unit. The conversion to cfs will be made in step 5.

2. Develop an elevation-storage curve for the structure.
The curve used in Example 17-1 will be used here. That curve is for storage in inches. The conversion to cfs-days will be made in step 5.

3. Develop and plot the inflow hydrograph.
Because of the type of problem the inflow hydrograph must be a Principal Spillway Hydrograph (PSH) taken from Chapter 21. The PSH corresponding to the PSMC of Example 17-1 is given in columns 1 and 4 of Table 21-7. The PSH is plotted in Figure 17-8.

4. Select the routing interval.
Examination of the PSH in Figure 17-8 shows that two routing intervals will be needed, one of 0.5 days for small changes in rates and one of 0.1 days for large changes.

5. Prepare the working curves.
Data and computations for the working curves are shown in Table 17-11. Two curves are needed because two routing intervals will be used. The elevations of column 1 and discharges of column 2 are taken from the curve of step 1 with the discharges being converted from in./hr. to cfs in the process. The discharges are selected so that they adequately define the elevation-discharge relationship. Column 3 of Table 17-11 gives the corresponding storages from the curve of step 2, converted from inches to cfs-hrs during the tabulation. The remaining columns contain self-explanatory computations. Columns 2 and 6 give the first working curve and columns 2 and 8 the second; they are plotted in Figure 17-9. Note that "lines of equal values" if drawn would be well above the working curves, therefore the routing intervals are adequately small. Also note that the second curve is shown only for the higher discharges in order to use a larger scale; ordinarily the entire curve is plotted.

Table 17-8 Working table for preparation of the working curve for Example 17-4.

O_2 (cfs)	$\frac{O_2}{2}$ (cfs)	S_2 (cfs-hrs)	$\frac{S_2}{\Delta t}$ (cfs)	$\frac{S_2}{\Delta t} + \frac{O_2}{2}$ (cfs)
(1)	(2)	(3)	(4)	(5)
0	0	0	0	0
50	25	70	140	165
150	75	164	328	403
300	150	248	496	646
800	400	651	1302	1702
1500	750	1302	2604	3354
3500	1750	3300	6600	8350
5000	2500	4540	9080	11580
7000	3500	5620	11240	14740
10000	5000	7130	14260	19260

Table 17-9 Operations table for the S-I method for Example 17-4.

Time (hrs)	$\bar{I}$ (cfs)	$\frac{S_2}{\Delta t} + \frac{O_2}{2}$ (cfs)	O (cfs)
(1)	(2)	(3)	(4)
0	0	0	0
.5	625*	625	285
1.0	1875	2215	1030
1.5	3125	4310	1880
2.0	4375	6805	2880
2.5	4615	8540	3610
3.0	3865	8795	3710
3.5	3125	8210	3450
4.0	2375	7135	3050
4.5	1635	5720	2440
5.0	900	4180	1810
5.5	265	2635	1210
6.0	0**	1425	630
6.5	0	795	375
7.0	0	420	160
7.5	0	260	82
8.0	0	178	53
etc.	etc.	etc.	etc.

* 625 cfs is the average discharge for the time from 0 to 0.5 hours, 1875 cfs the average discharge from 0.5 to 1.0 hours, and so on.

** Inflow ceases at 5.33 hours.

Table 17-10 Procedure for routing by the Storage-Indication method for Example 17-4.

Time (hrs)	$\bar{I}$ (cfs)	$\frac{S_2}{\Delta t} + \frac{O_2}{2}$ (cfs)	O (cfs)	Remarks
(1)	(2)	(3)	(4)	(5)
0	0	0	0	Given
.5	625			Given
		625		0 - 0 + 625 = 625
			285	From Figure 17-6
1.0	1875			Given
		2215		625 - 285 + 1875 = 2215
			1030	From Figure 17-6
1.5	3125			Given
		4310		2215 - 1030 + 3125 = 4310
			1880	From Figure 17-6
2.0	4375			Given
		6805		4310 - 1880 + 4375 = 6805
			2880	From Figure 17-6
etc.	etc.	etc.	etc.	etc.

6. Prepare the operations table.
Suitable headings and arrangement are shown in Table 17-12. Note that there is a column for instantaneous rates of inflow. These rates will be used for getting $\bar{I}$ values because it is difficult to select $\bar{I}$ values accurately enough from some portions of the plotted hydrograph.

7. Tabulate times and rates of inflow and compute $\bar{I}$ values.
Accumulated times are shown in column 1 of Table 17-12 at intervals of $\Delta t = 0.5$ days for the initial slow-rising portion of the PSH, at $\Delta t = 0.1$ days for the fast-rising and -falling portion, and again at $\Delta t = 0.5$ days for the slow recession. Instantaneous rates of inflow for those times are taken from the PSH of Figure 17-8 (or from column 4 of Table 21-7 if they are for the selected times) and shown in column 2. The $\bar{I}$ values of column 3 are arithmetic averages of entries in column 2.

8. Do the routing.
The procedure is the same as that given in Table 17-10 except when a change is made from one working curve to another. The changes are made as follows. At time 4.5 days the routing interval changes, therefore, the working curve must be changed. The outflow rate at that time is 116 cfs. Entering the second working curve with this rate gives 2,640 cfs as the value of $(S_2/\Delta t) + (O_2/2)$ in column 4 for the same time. Once this value is entered the routing continues with use of the second working curve. At time 6.0 days the routing interval changes back to the first one and therefore the first working curve must again be used. The outflow rate at that time is 357 cfs. Entering the first working curve with this rate gives 1,270 cfs as the value of $(S_2/\Delta t) + (O_2/2)$ in column 4 for that time. After entering this value the routing continues with use of the first working curve.

9. Determine the maximum storage attained in the routing.
The maximum storage attained in a reservoir during the routing of a single-peaked hydrograph occurs at the time when outflow equals inflow. The plotting in Figure 17-8 shows that this occurs at 5.33 days. For this time, Table 17-12 shows that $O_2 = 364$ cfs and $(S_2/\Delta t) + (O_2/2) = 6{,}480$ cfs. Solving for S_2 gives $S_2 = \Delta t\ 6480 - (O_2/2)$. With $\Delta t = 0.1$ days and $O_2 = 364$ cfs, $S_2 = 0.1\ 6480 - (364/2) = 629.8$ cfs-days, the maximum storage. To convert to AF use Equation 17-2, which gives $629.8/0.504 = 1{,}247$ AF as the maximum storage in AF. To convert AF to inches use Equation 17-3 and the given drainage area of 8.0 square miles (see Example 17-1), which give $1247/53.3(8.0) = 2.93$ inches as the maximum storage in inches. (Note: The storage can also be found by use of a storage-discharge curve or elevation-discharge and elevation-storage curves but with the Storage-Indication method it is generally best to use the above method.)

A comparison of peak rates of outflow shows that the mass-curve method of Example 17-1 gave a peak rate of 1.69 in./day, which converts to 363

Table 17-11 Working table for preparation of the working curves for Example 17-5.

Elevation	Discharge (O_2)	Storage (S_2)	$\frac{O_2}{2}$	For Δt = 0.5 days: $\frac{S_2}{\Delta t}$	For Δt = 0.5 days: $\frac{S_2}{\Delta t} + \frac{O_2}{2}$	For Δt = 0.1 days: $\frac{S_2}{\Delta t}$	For Δt = 0.1 days: $\frac{S_2}{\Delta t} + \frac{O_2}{2}$
(feet)	(cfs)	(cfs-days)	(cfs)	(cfs)	(cfs)	(cfs)	(cfs)
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
580.2	0	0	0	0	0	0	0
581.2	11.6	47.0	6	94	100	470	476
582.2	32.8	96.5	16	193	209	965	981
583.3	60.3	165	30	330	360	1650	1680
584.6	108	236	54	472	526	2360	2414
586.0	133	324	66	648	714	3240	3306
587.0	149	393	75	786	861	3930	4005
587.5	204	431	102	862	964	4310	4412
588.0	289	471	144	942	1086	4710	4854
588.5	353	512	176	1024	1200	5120	5296
590.0	365	643	182	1286	1468	6430	6612
592.0	382	832	191	1664	1855	8320	8511
595.0	401	1165	200	2330	2530	11650	11850

Table 17-12 Operations table for Example 17-5.

Time (days)	Inflow (cfs)	$\bar{I}$ (cfs)	$\frac{S_2}{\Delta t} + \frac{O_2}{2}$ (cfs)	Out-flow (cfs)
(1)	(2)	(3)	(4)	(5)
0	0	0	0	0
.5	70	35	35	3
1.0	79	74	106	12
1.5	84	82	176	28
2.0	88	86	234	38
2.5	99	94	290	48
3.0	110	104	346	57
3.5	128	119	408	72
4.0	156	142	478	94
4.5	245	200	584*	116
			2640**	116
4.6	269	257	2781	119
4.7	308	288	2950	123
4.8	380	344	3171	129
4.9	522	451	3493	137
5.0	2002	1262	4618	240
5.1	1049	1526	5904	359
5.2	577	813	6358	363
5.3	393	485	6480	364
5.4	312	352	6468	364
5.5	267	290	6394	363
5.6	217	242	6273	362
5.7	200	208	6119	361
5.8	184	192	5950	360
5.9	174	179	5769	358
6.0	164	169	5580**	357
			1270*	357
6.5	138	146	1059	266
7.0	118	128	921	175
7.5	106	112	858	148
8.0	94	100	810	142
etc.	etc.	etc.	etc.	etc.

* From first working curve.
**From second working curve.

cfs, and the Storage-Indication method gave 364 cfs, which is excellent agreement. But a comparison of maximum storage in inches shows that the mass-curve method of Example 17-1 gave 2.82 inches, the graphical mass-curve method of Example 17-3 gave 2.80 inches, and the Storage-Indication method gave 2.93 inches. The discrepancy is for the most part due to use of small-scale graphs for the working curves. Larger graphs would reduce the discrepancy.

Storage-Indication Method as Used in the SCS Electronic Computer Program.- SCS electronic computer program for watershed evaluations uses the Storage-Indication method only for reservoir routings. The chief difference between the manual procedure of Example 17-5 and the electronic-computer procedure is that in the latter no working curves are used. Instead, the working equation is solved during a process in which interpolations are made in the elevation-discharge and elevation-storage data stored in the computer. The process is repeated during the routing just as the working curve is repeatedly used in manual routing. The machine routing has a numerical accuracy greater than that of the manual routing, but the machine cannot improve the accuracy of the input data. Details of the machine routing process are given in pages A-61 through A-66 of the report titled "Computer Program for Project Formulation - Hydrology," by C-E-I-R, Inc. Arlington, Va., January 1964, which was prepared for SCS. Copies of this report are available from the Washington, D. C. office of SCS.

Culp's Method.- Some routing methods are developed for solving special problems, for which they have a high efficiency. One such method is described next.

In the design of an emergency spillway of a dam it is SCS practice to base the design on the results from a routing of an Emergency Spillway Hydrograph. Because all of the spillway dimensions cannot be known in advance, it is necessary to route the hydrograph through three or four different spillways with assumed dimensions before the spillway with the proper dimensions can be found. M. M. Culp's routing method eliminates much of that work by giving the routed peak discharge without the use of spillway dimensions. The following example shows an application of the method to the structure used in previous examples. The example is lengthy because many details are given; after the method is understood it will be seen to be fast and easy to apply.

Example 17-6.--Find the routed peak discharge to be used in design of an emergency spillway for the structure of Example 17-1. The required difference in elevation between the crest of the spillway and the reservoir water surface, H_p, is 4.0 feet during the peak discharge. Watershed and structure data are given in examples 17-1 and 17-2.

1. Prepare the elevation-discharge curve for the principal spillway.
This curve was prepared for Example 17-1 with the discharges in inches per hour. It will be used here as shown in Figure 17-10(a) with discharges in cfs.

2. Prepare the elevation-storage curve for the structure.
This curve was prepared for Example 17-1. Only the portion above the sediment storage will be used here; it is shown in figure 17-10(a).

3. Determine the elevation of the emergency spillway crest.
According to SCS criteria, the elevation of the emergency spillway crest can be at or above the maximum water-surface elevation attained in the reservoir during the routing of the Principal Spillway Hydrograph (PSH) or its mass curve (PSMC). The water-surface elevation found in Example 17-1 will be used here as the crest elevation. This elevation is 589.5 feet with floodwater storage of 2.82 inches.

4. Determine the water-surface elevation of the floodwater remaining in the reservoir after 10 days of drawdown from storage at the water-surface elevation attained in routing the PSH or PSMC.
This step is required by SCS criteria. The determination is made in Example 17-2 and those results will be used here. The water-surface elevation after 10 days of drawdown is 581.1 feet with floodwater storage at 0.20 inches.

5. Prepare the Emergency Spillway Hydrograph (ESH) and its mass curve (ESMC).
The ESH for this example was prepared using the method of Example 21-5 and the following data: drainage area = 8.0 square miles, time of concentration = 2.0 hours, runoff curve number = 75, design storm rainfall = 9.1 inches, storm duration = 6.0 hours, runoff = 6.04 inches, hydrograph family = 2, T_o = 5.05 hours, initial T_p = 1.4 hours, T_o/T_p = 3.61, selected T_o/T_p = 4, revised T_p = 1.26 hours, q_p = 3,073 cfs, and $Q(q_p)$ = 18,560 cfs. The ESMC was prepared using Table 21-17 and the following data: hydrograph family = 2, T_o/T_p = 4, T_p = 1.26 hours, and Q = 6.04 inches. The hydrograph is shown in Figure 17-10(b) and the mass curve in Figure 17-10(c).

(Note: The above steps are taken, in much the same way, regardless of which manual method of routing is used for this kind of problem. The following steps apply to the Culp method.)

6. Determine the time at which the emergency spillway begins to flow during passage of the ESH or ESMC.
For this example the time was found by routing the ESMC of step 5 by the method of Example 17-1, using the curves of Figure 17-10(a) as working curves. The routing was started with 0.20 inches of floodwater in the reservoir (SCS criteria require the ESH or ESMC routing to start at the elevation for the floodwater remaining after the 10-day drawdown period; see step 4). The emergency spillway began to flow at 2.9 hours, at which time the mass outflow was 0.06 inches. The time and outflow are indicated by point c1 on Figure 17-10(c)

<u>7. Determine the average discharge of the principal spillway during passage of the ESH or ESMC through the emergency spillway.</u> The principal spillway average discharge is for the period during which the reservoir storage rises from the elevation of the emergency spillway crest to the crest elevation plus H_p. Use the elevation-discharge curve of Figure 17-10(a) to find the discharges at the two elevations. These discharges are 361 and 392 cfs respectively; their average is 376 cfs.

<u>8. Locate a reference point in the ESH for use in later steps.</u> The reference point, shown as point b1 in Figure 17-10(b), is located at the time determined in step 6 and at the average discharge determined in step 7. A second point, not actually necessary in the work, is shown as b2 on the recession side. A straight line connecting points b1 and b2 represents the principal spillway outflow rate during the period used in step 7.

<u>9. Compute the slope of the principal spillway mass outflow line for use on the mass inflow graph.</u> The mass outflow to be used is for the period considered in step 7. Full pipe flow occurs and the mass outflow is adequately represented by a straight line. The slope of the line for this example must be in inches per hour because the mass inflow scales are for inches and hours. To get the slope, convert the average discharge of step 7 by use of Equation 17-5, which gives 376/645(8.0) = 0.073 inches per hour.

<u>10. Plot a reference line and a working line of principal spillway mass outflow on the graph for mass inflow.</u> The lines are for the period considered in step 7 but for working convenience they are extended beyond the limits of the period. To plot the reference line, first locate point c2 on the mass inflow curve of Figure 17-10(c) at the time determined in step 6, then through c2 draw a straight line having the slope determined in step 9; this gives line A as shown. To plot the working line, first determine the storage associated with H_p, which is 1.84 inches as shown in Figure 17-10(a), then draw line B parallel to line A and 1.84 inches of runoff above it as shown in Figure 17-10(c).

<u>11. Find the period within which the emergency spillway peak discharge will occur.</u> Point c3 is at the intersection of the mass inflow curve and line B in Figure 17-10(c). Locate point b3 on the ESH of Figure 17-10(b) at the time found for c3. Points b3 and b2 are the end points for the period within which the emergency spillway peak discharge will occur.

<u>12. Select several working discharges between points b3 and b2.</u> Four selected working discharges are indicated by points b4, b5, b6, and b7 in Figure 17-10(b); the discharges are 4,750, 3,500, 2,200, and 920 cfs respectively. These discharges represent the peak discharges of outflow hydrographs.

(Note: After some experience with this method, it may be found easier to select only two working discharges in this step, to work through steps 13 to 15, and if the results are unsatisfactory to return to step 12 again by selecting a third working discharge, working through steps 13 through 15 for that discharge, and so on.)

13. Compute a volume-to-peak for each working discharge of step 12. In the Culp method the rising side of the outflow hydrograph for a trapezoidal spillway is taken as being nearly parabolic so that the volume from the beginning of rise to the peak rate, or the volume-to-peak, is:

$$Qe = 0.62\ (qe - qps)\ Te \qquad \text{(Eq. 17-18)}$$

where Qe is the volume in cfs-hrs, qe is the working discharge of step 12 in cfs, qps is the principal spillway rate of step 7 in cfs, and Te is the time in hours from point b1 to the peak time. The volume Qe must be converted to a unit usable with the mass inflow curve, in this case, inches. The summary of work for this step is given in Table 17-13. In the columns for points b4 through b7, the items in line 1 are from step 12; items in line 2 are from step 7; items in line 3 are obtained by subtracting qps from qe; items in line 4 are obtained by inspection of Figure 17-10(b); items in line 5 are products of (qe - qps) x Te; items in line 6 are products of (Qe/0.62) x 0.62; items in line 7 are Qe's of line 6 divided by the drainage area of 8.0 square miles; items of line 8 are Qe's of line 7 divided by 645. Each Qe of line 8 applies only at the time indicated by its point on the ESH.

14. Plot a curve of mass inflow minus mass outflow. This is a working curve, not the complete curve of inflow minus outflow. Subtract each Qe of line 8, Table 17-13, from the inflow amount at the identical time on the mass inflow curve of Figure 17-10(c) and plot the result as shown for points c4, c5, c6, and c7. Connect the points with a curve, line C.

15. Determine the time and rate for the emergency spillway peak discharge. The intersection of lines B and C, at point c8 in Figure 17-10(c), gives the time at which the emergency spillway peak discharge occurs The total discharge rate at that time is 3,050 cfs as shown by the corresponding point b8 on the ESH of Figure 17-10(b). The emergency spillway discharge rate is 3050 - 376 = 2,674 cfs, which occurs when the reservoir water surface is at the given elevation of 593.5 feet (crest elevation plus H_p). This step completes the routing. Design of the emergency spillway now follows with use of ES-98, ES-124, and spillway criteria.

If H_p is not known in advance, the Culp method can be used with assumed values of H_p to get associated discharges from which the suitable combination of H_p and discharge can be selected. For earth spillways H_p can be closely approximated from permissible velocities and the appropriate

Table 17-13 Working table for Culp method step 13 of Example 17-6.

Line	Item	Unit	Point: b4	b5	b6	b7
1	qe	cfs	4750	3500	2200	920
2	qps	cfs	376	376	376	376
3	qe - qps	cfs	4374	3124	1824	544
4	Te	hrs	2.1	2.8	3.4	4.1
5	Qe/0.62	cfs-hrs	9180	8750	6200	2230
6	Qe	cfs-hrs	5680	5420	3840	1380
7	Qe	csm-hrs	710	677	480	173
8	Qe	in.	1.102	1.050	0.745	0.268

length and chosen profile of the inlet channel. A close approximation of the emergency spillway discharge rate can be obtained in this way for an H_p value near the middle of the desired range to get a "C curve" (line C on Figure 17-10(c)). The average discharge in the conventional drop inlet under full pipe flow conditions varies only slightly as H_p varies relatively greatly, thus the discharge through the emergency spillway can be closely approximated from such an average C curve. If refinement is justified, then trial adjustments on the slope of line B will give the required accuracy. The correction process converges rapidly. For preliminary layouts or comparative cost studies such refinement is seldom justified.

Short-Cuts for Reservoir Routings.- Various equations and charts have been developed for quickly estimating the required storage in a reservoir or the required capacity of a spillway, such estimates being used in preliminary studies of structures or projects. The equations and charts are usually based on the results of routings so that using the equation or chart is in effect a form of routing.

A typical short-cut is the graph, Figure 17-11. The curve through the circled points is based on information in table 2 on page 39 of "Low Dams," a design manual prepared by the Subcommittee on Small Water Storage Projects, National Resources Committee, Washington, D. C., 1938 (the manual is out of print and no longer available for purchase). Relationships of this kind are developed from routings made through a particular type of spillway and they apply only to that type. The form of standard inflow hydrograph used for routing also affects the relationship and the same form must be applicable when the short-cut is used. With such a relationship if any three of the four variables are known the fourth can be estimated. Usually either the reservoir storage or the reservoir discharge rate is the unknown.

The triangular point on Figure 17-11 is for the routing made in Example 17-6. For that example the outflow/inflow ratio is 3050/10200 = 0.30 and the storage/inflow-volume ratio is 2.82/(2.62 + 1.84) = 0.63. Note that the emergency spillway "surcharge" storage is included when computing the volume ratio. The cross points, for "miscellaneous routings", are for routings of several kinds of hydrographs through emergency spillways of the SCS type. The "Low Dams" curve appears to be an enveloping curve for the points. As such it can be used for making conservative estimates. Thus, if the inflow volume is 8.15 inches of runoff and the total available storage is 5.7 inches then the storage ratio is 0.7; at that ratio the discharge ratio is 0.4, which means that the peak outflow rate will be not more than 0.4 of the peak inflow. Such estimates are often useful in preliminary work.

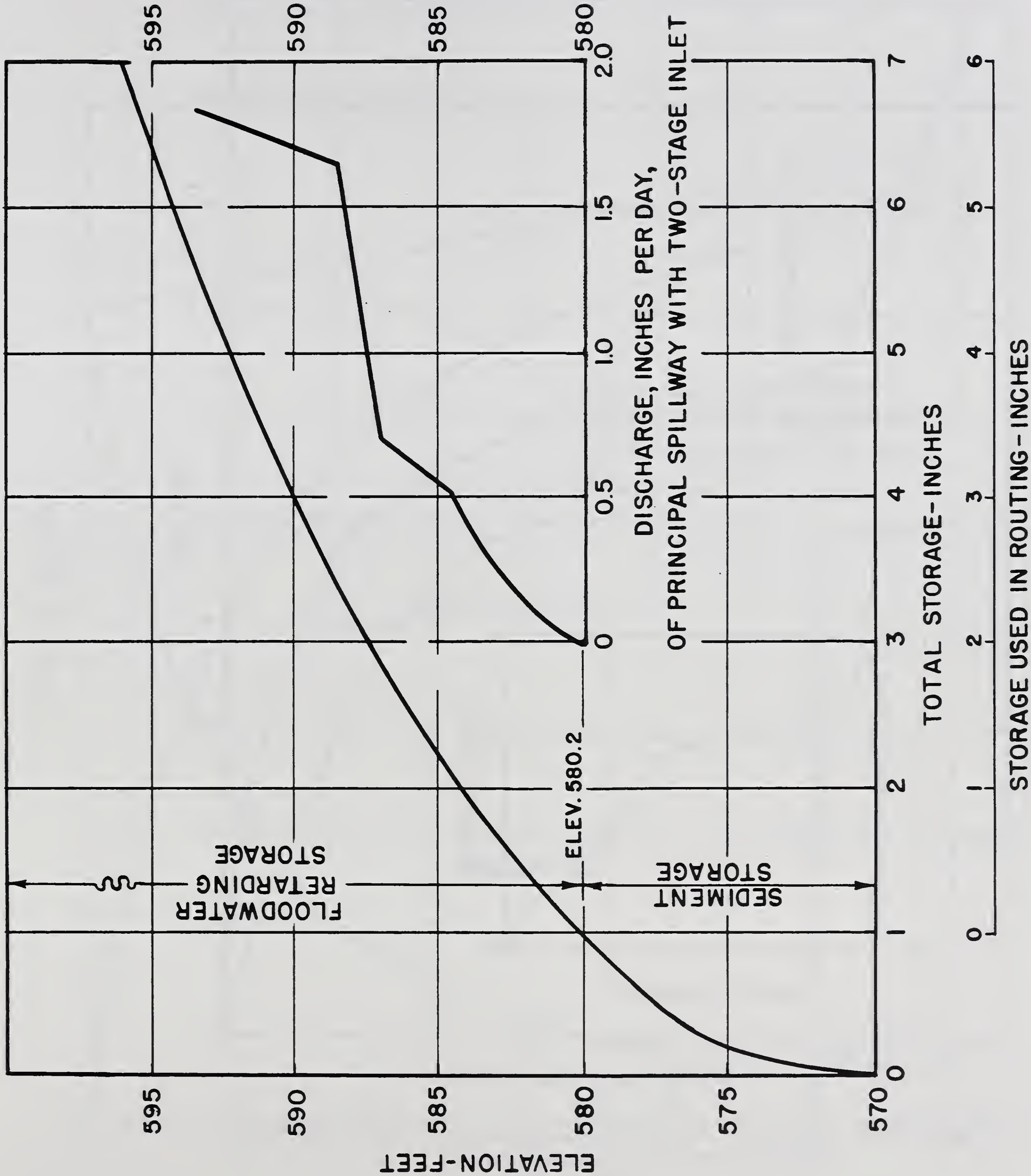


Figure 17-1. Elevation, storage, discharge relationship for a reservoir.

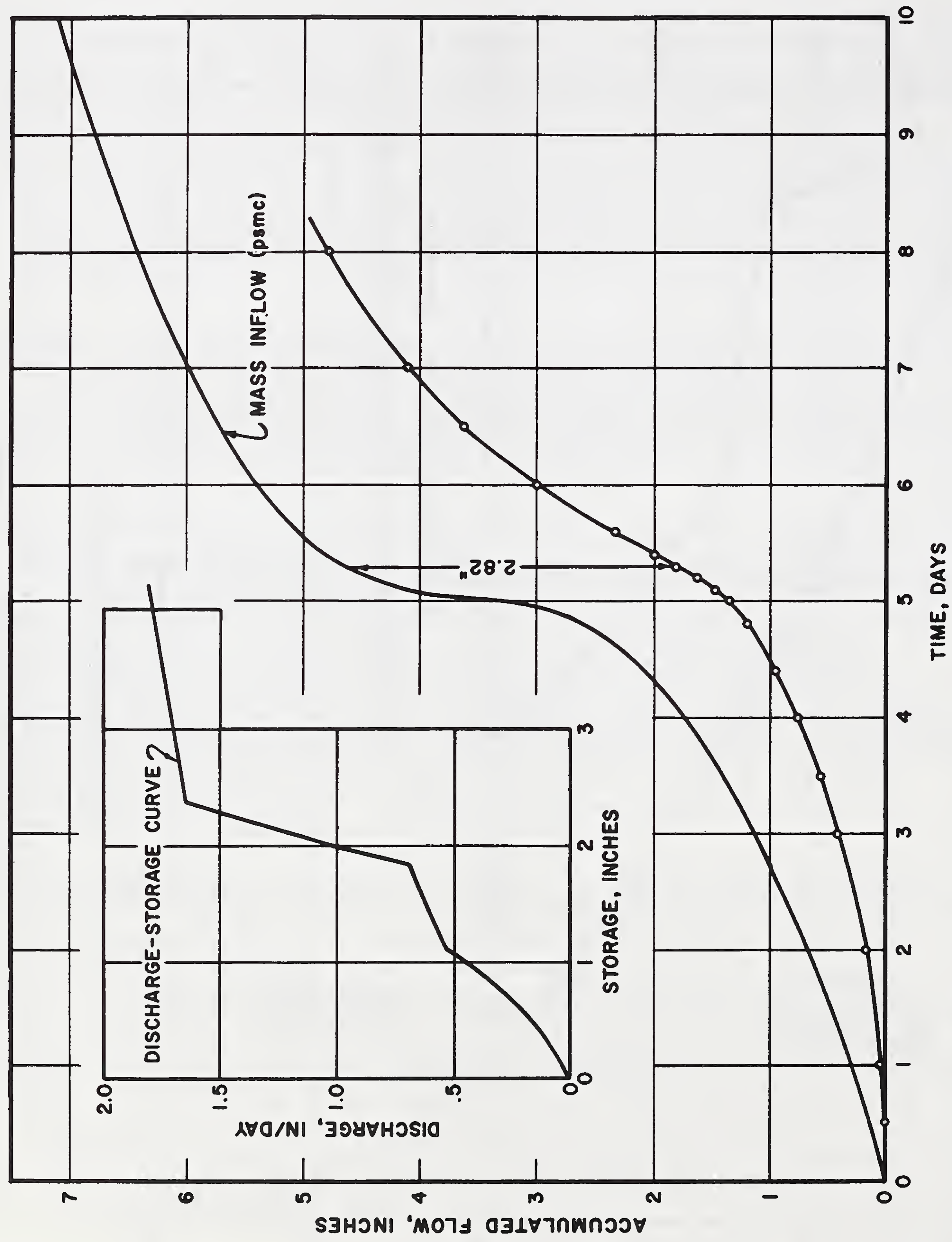


Figure 17-2. Storage, discharge relationship and plotted mass inflow curve for a reservoir.

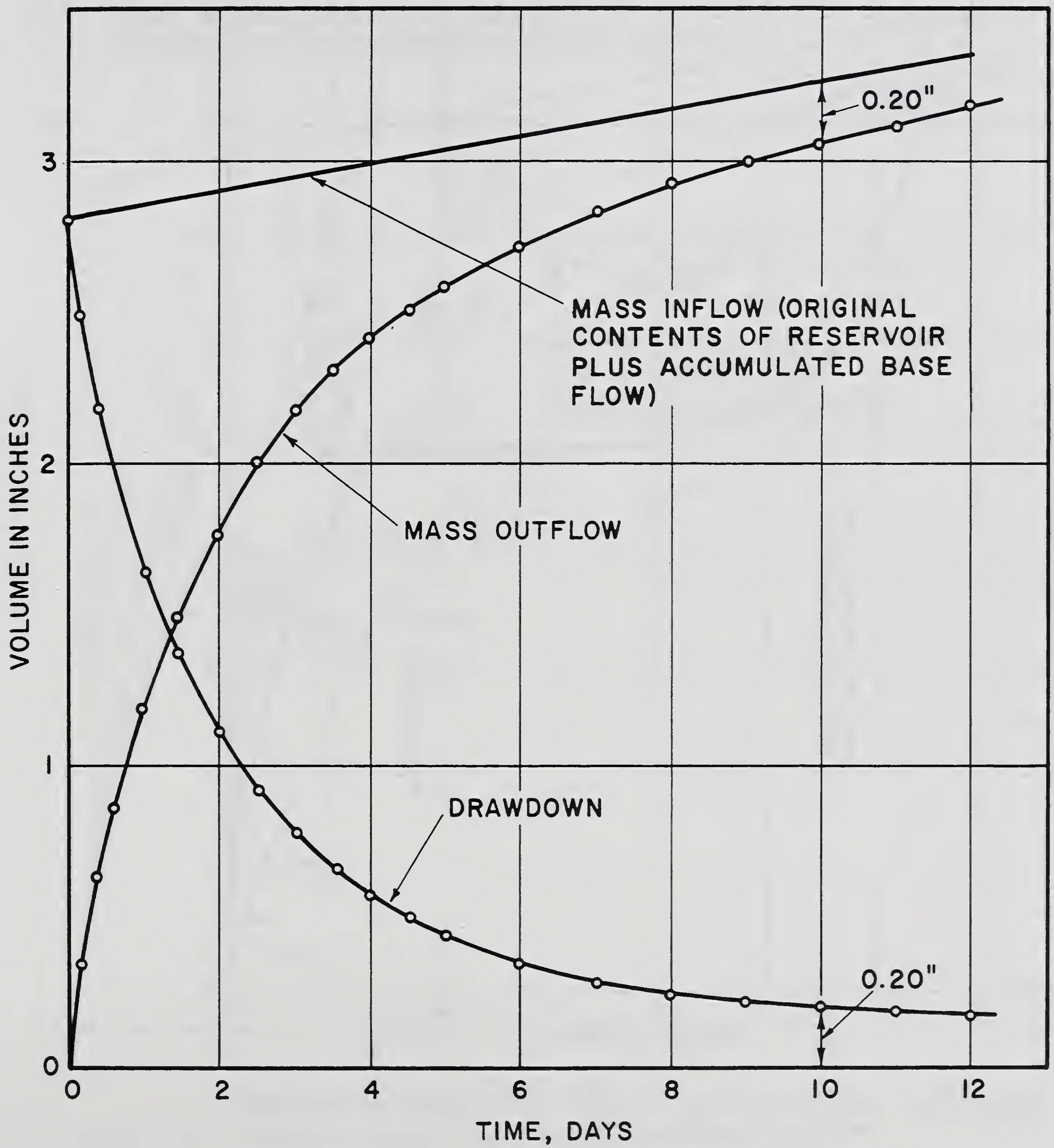


Figure 17-3. Mass inflow, storage, and mass outflow curves for Example 17-2.

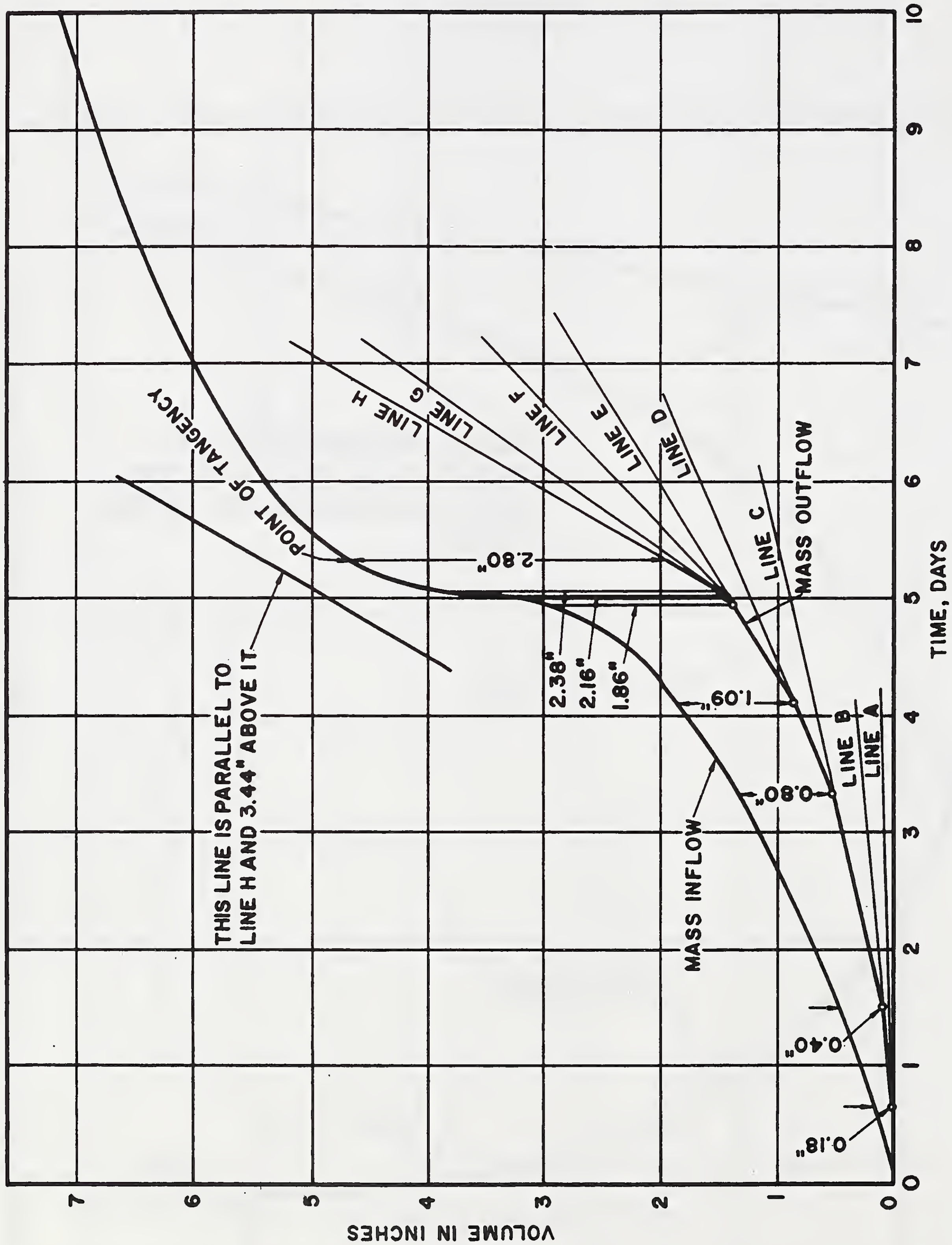


Figure 17-4. Graphical version of Mass Curve method of reservoir routing for Example 17-3.

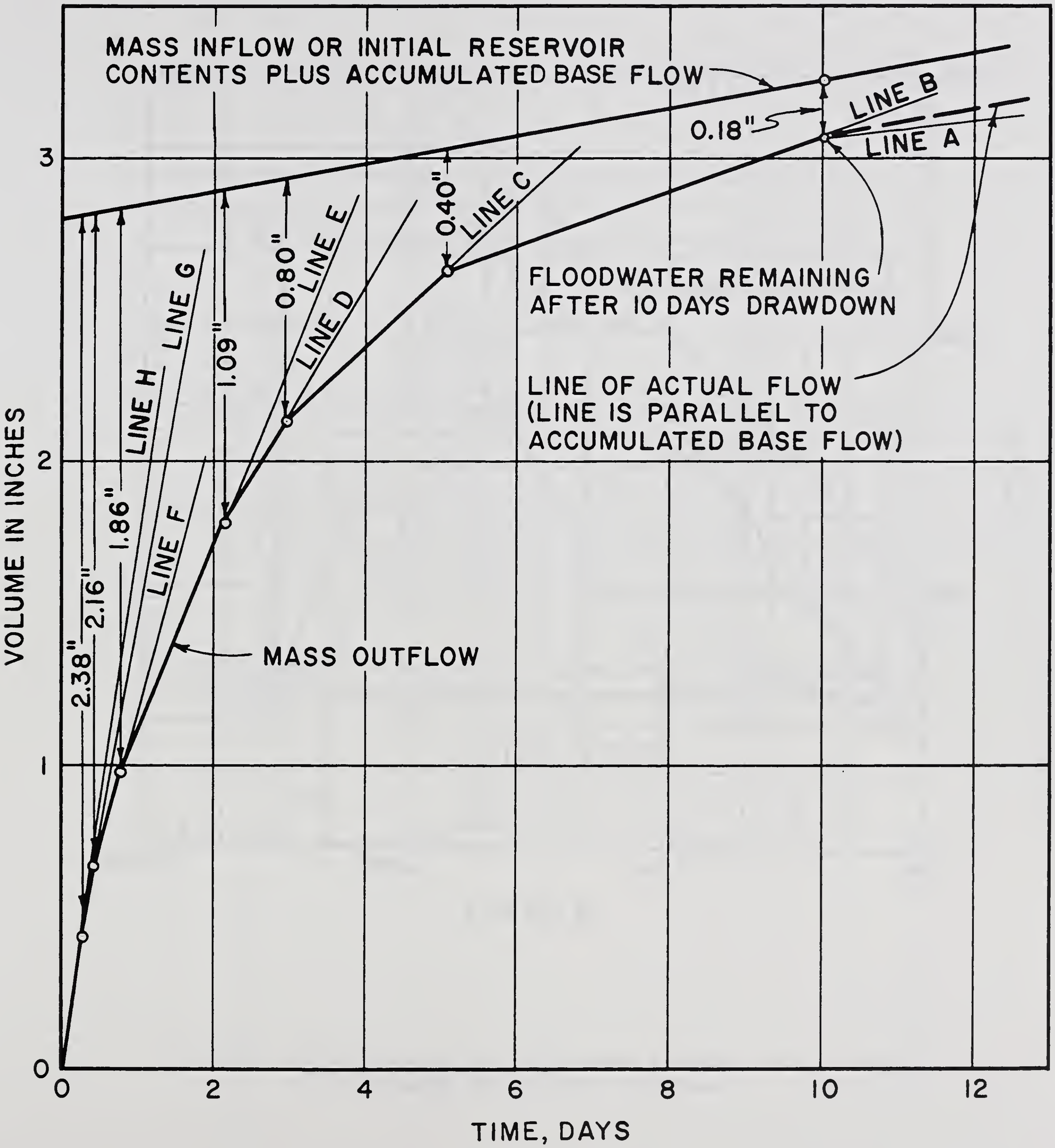


Figure 17-5. Graphical version for Example 17-2, Step 4.

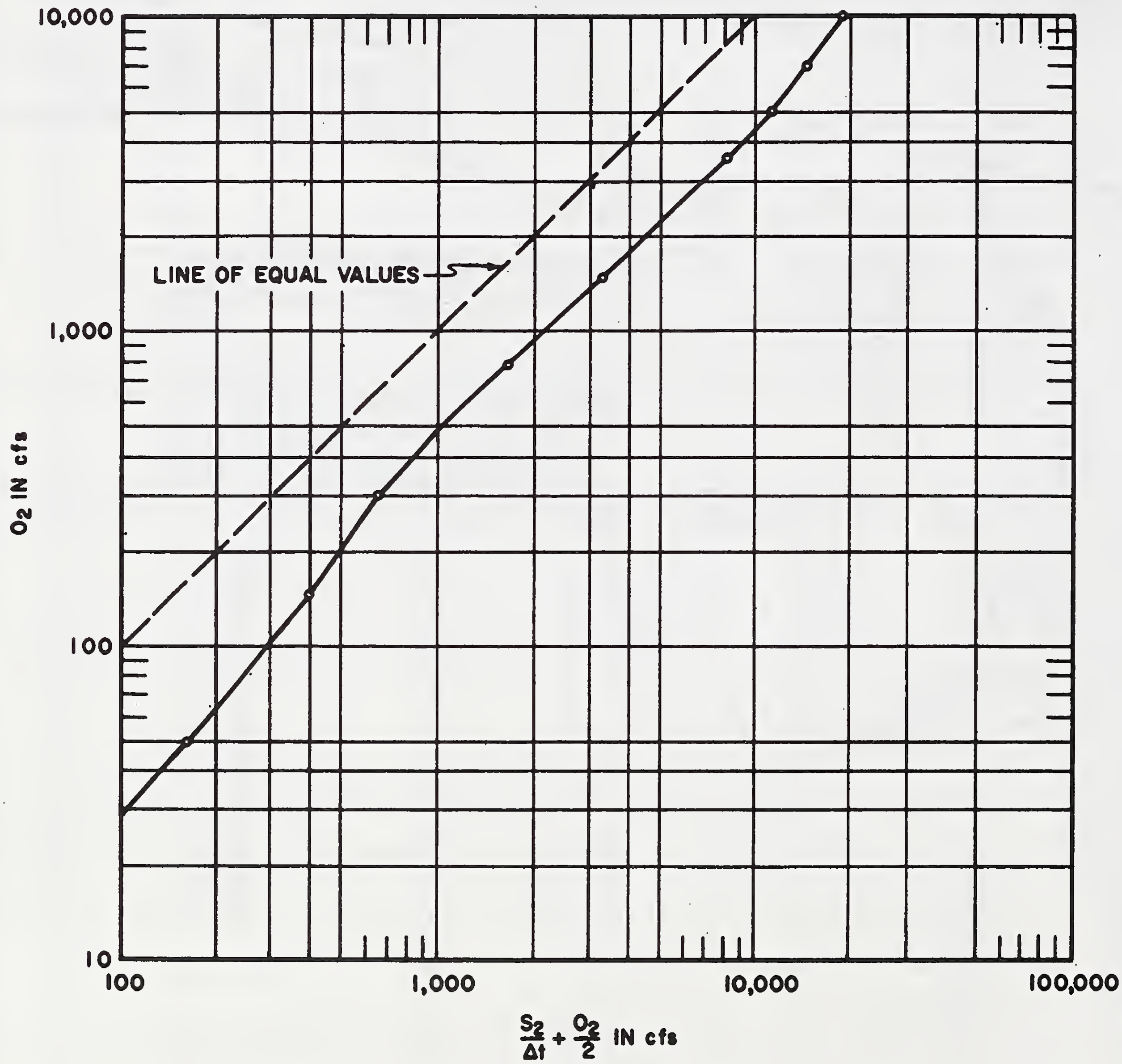


Figure 17-6. Working curve for Storage-Indication method of reservoir routing for Example 17-4.

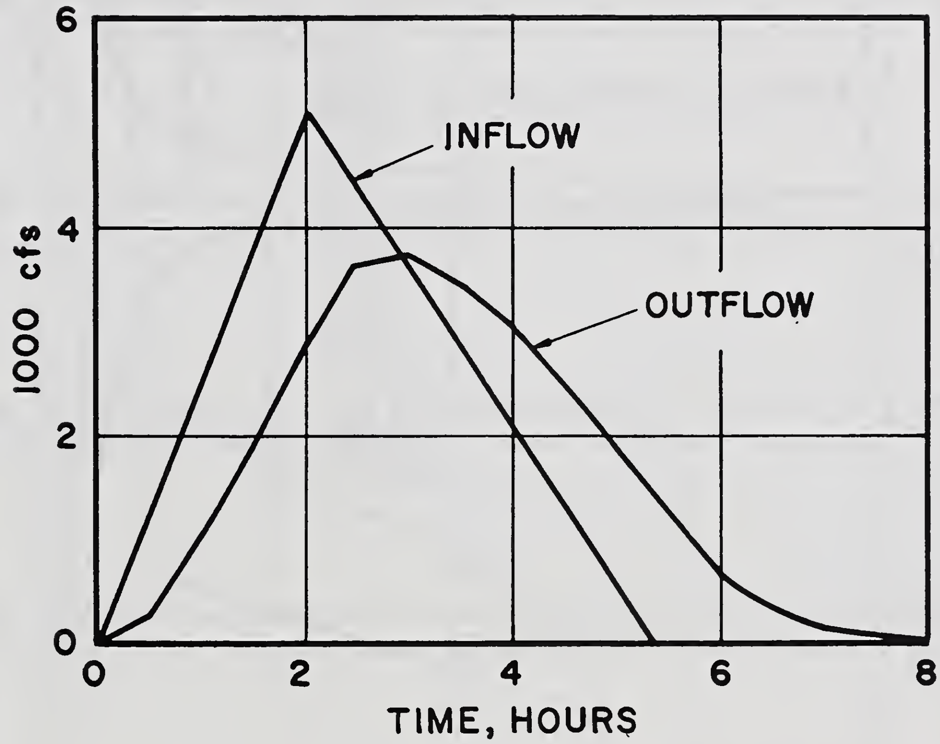


Figure 17-7. Inflow and outflow hydrograph for Example 17-4.

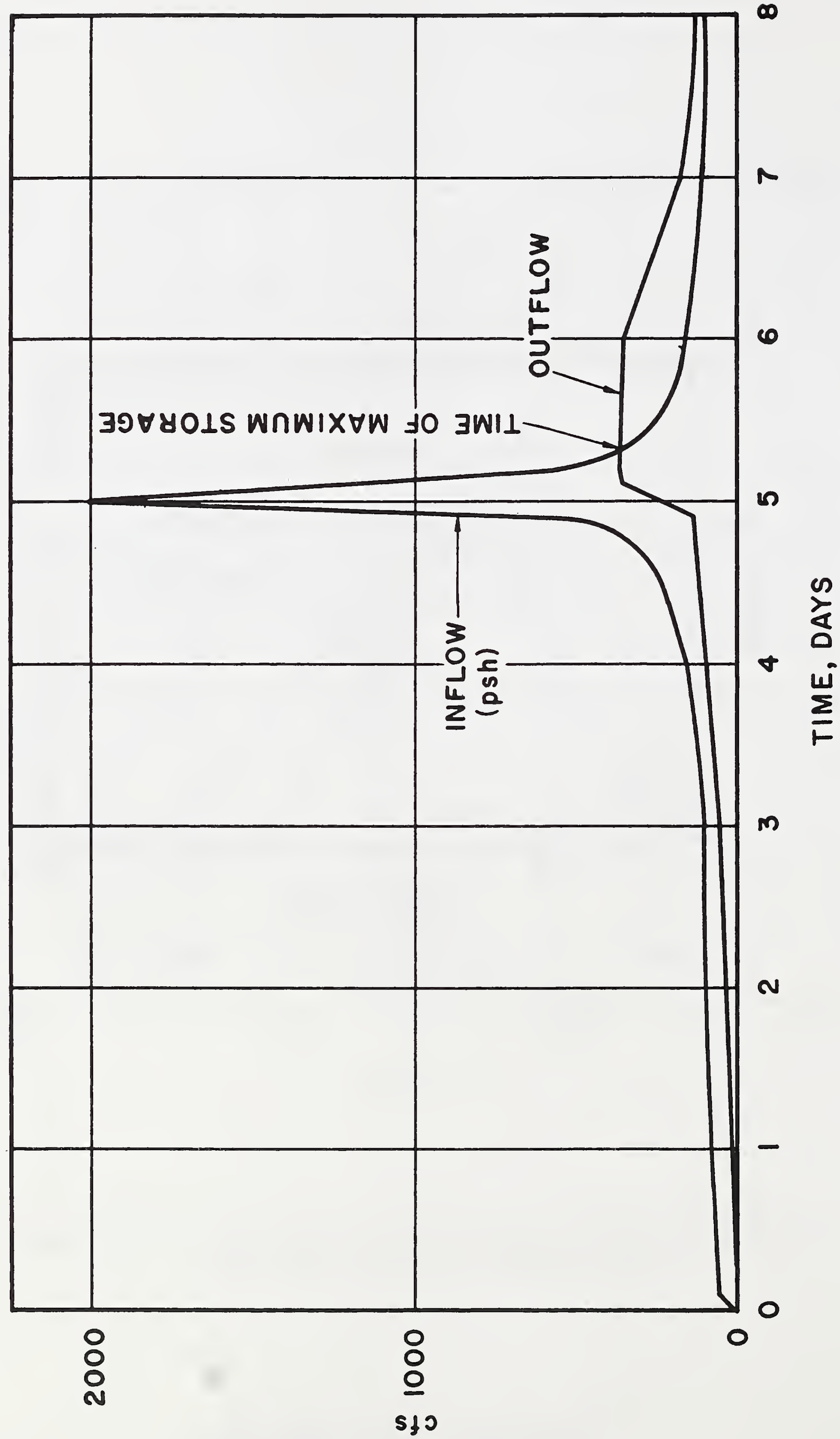


Figure 17-8. Principal spillway hydrograph and outflow hydrograph for Example 17-5.

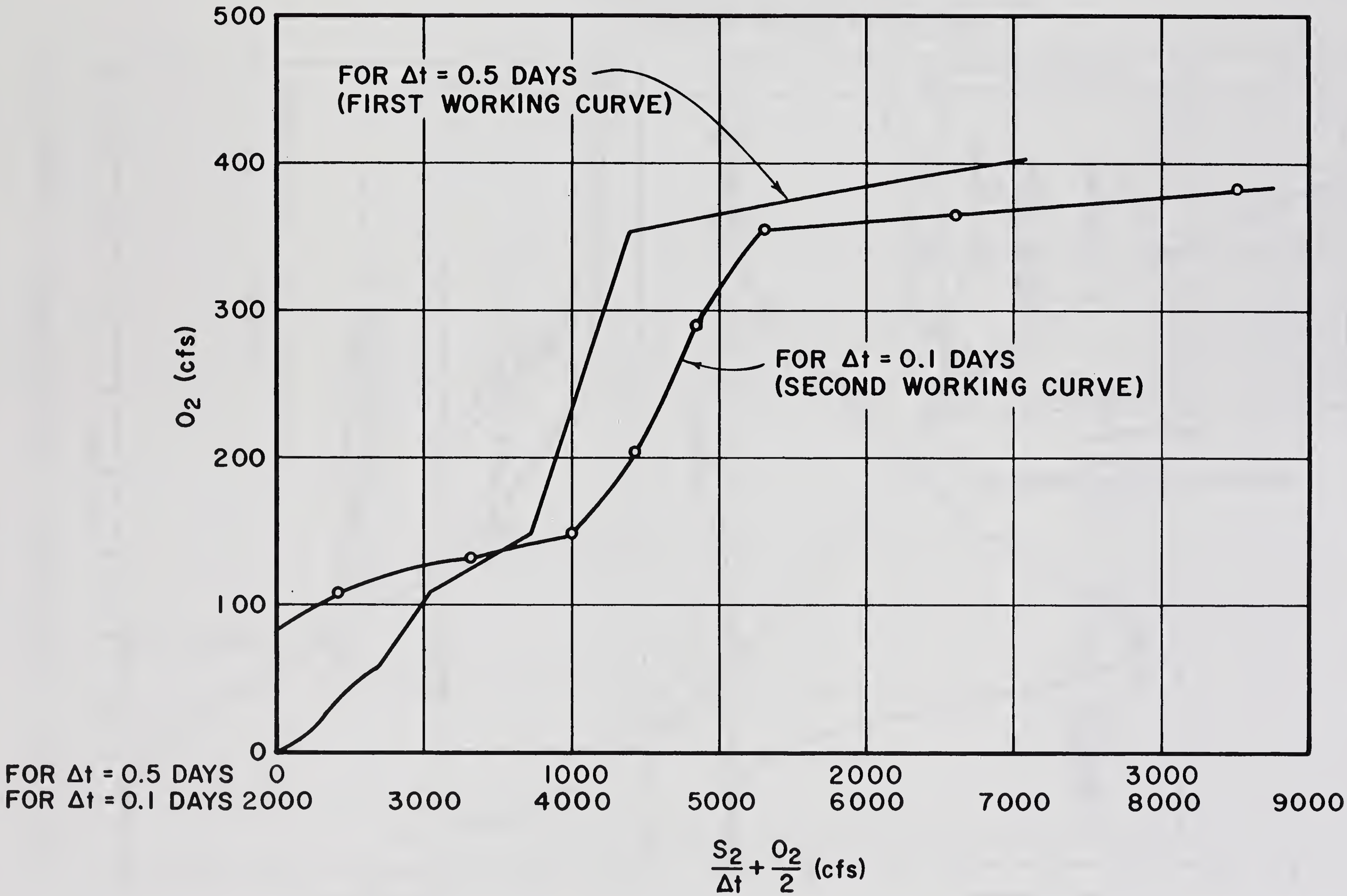


Figure 17-9. Working curves for Storage-Indication method of reservoir routing for Example 17-5.

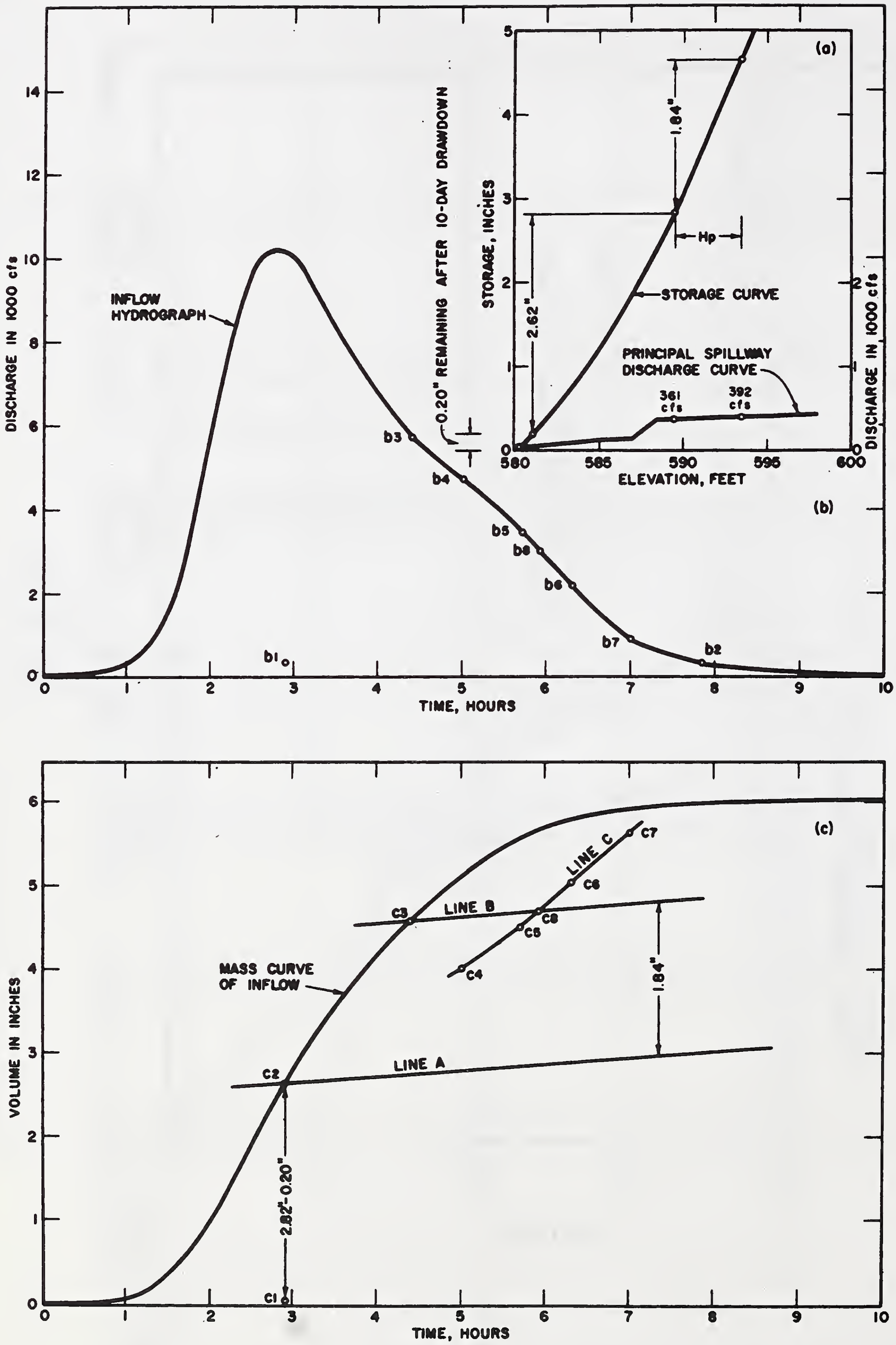


Figure 17-10. Culp's method of reservoir routing for Example 17-6.

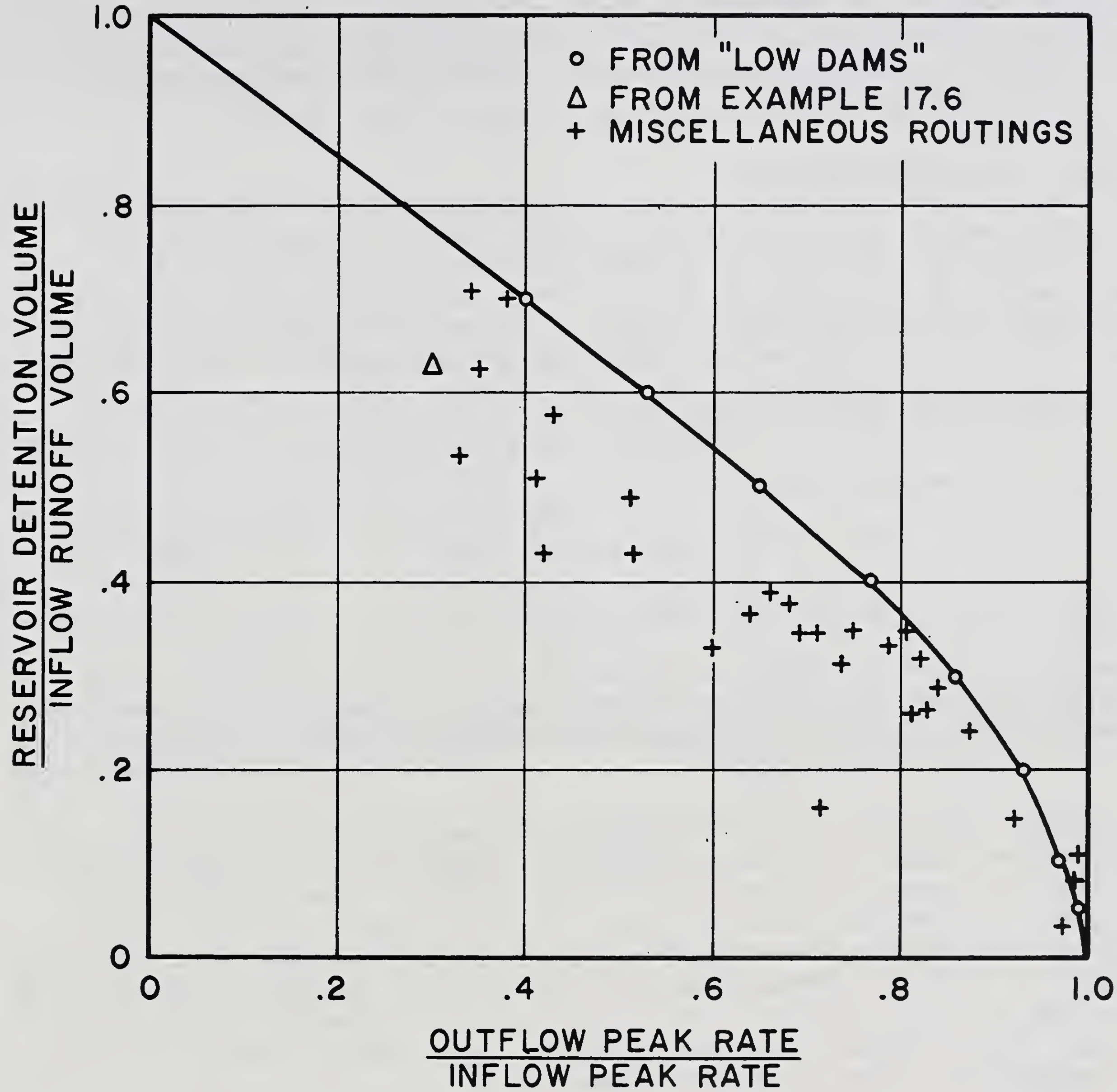


Figure 17-11. Typical shortcut method of reservoir flood routing.

Channel Routing Methods

The Convex method of routing through stream channels is presented in this part. The method is derived from inflow-outflow hydrograph relationships and, because of this, the method has some features not possessed by channel routing methods derived from consideration of the continuity equation. The Storage-Indication method of channel routing, presented in Example 17-4, will not be discussed here, but discussions of procedures for adding local inflows, deducting transmission losses, and routing through stream systems also apply to that method.

Theory of the Convex Method

The Convex method is based on the following principle: When a natural flood flow passes through a natural stream channel having negligible local inflows or transmission losses, there is a reach length L and a time interval Δt such that O_2 is not more than the larger nor less than the smaller of the two flows I_1 and O_1. Δt is considered as both the travel time of the flood wave through the reach measured at the beginning of the rising portion of the hydrograph at both ends of the reach; and the required routing time interval.

The principle requires that:

$$\text{If } I_1 \geq O_1 \text{, then } I_1 \geq O_2 \geq O_1 \qquad \text{(Eq. 17-19)}$$

$$\text{If } I_1 \leq O_1 \text{, then } I_1 \leq O_2 \leq O_1 \qquad \text{(Eq. 17-20)}$$

In general, inequality Equation 17-19 applies to rising portions of hydrographs and Equation 17-20 to falling portions. Note that I_2 does not enter into the principle; this makes the Convex method a forecasting method (see under "Discussion").

The routing principle can be extended to include local inflows and transmission losses but this unnecessarily complicates the working equation. It is common practice to add local inflows to the routed outflow hydrographs to get the total outflows, and this practice will be followed here. There may be situations, however, in which the local inflow is added to the inflow hydrograph and then routed. Small transmission losses are generally deducted after the routing, large ones during the routing; for a discussion of transmission losses see the heading "Effects of transmission losses on routed flows."

The routing or working equation is formed after examination of typical inflow and outflow hydrographs such as those in Figure 17-12. Typical flood wave combinations of I_1, O_1 and O_2 are shown on the rising and falling sides of the hydrographs. The routing principle states that for a properly selected reach length L, hence Δt, O_2 will fall somewhere on or between I_1 and O_1 in magnitude but not above or below them. This is evident on Figure 17-12 despite the displacement of O_2 in time; it is the magnitudes that are of concern here.

The next step is to recognize that I_1, O_1 and O_2 are members of a Convex set[1/]. For such a set, if points A and B are in the set then all points on a straight line connecting A and B are also in the set. Because the concern is with magnitudes and not with time it is not necessary for O_2 to be physically on the line between I_1 and O_1. The routing equation can now be written based on the theory of convex sets. For the situation just described, and using proportions as shown on the inset of Figure 17-12, the routing or working equation is:

$$O_2 = (1-C)O_1 + C\,I_1 \qquad \text{(Eq. 17-21)}$$

where C is a parameter with the range:

$$\text{zero} \leq C \leq \text{one} \qquad \text{(Eq. 17-22)}$$

Given Equation 17-22, Equation 17-21 meets the requirements of Equations 17-19 and 17-20 and therefore of the routing principle.

The routing method based on Equation 17-21 is called the Convex method to call attention to the equation's background.

It follows from Equation 17-21 that:

$$C = \frac{O_2 - O_1}{I_1 - O_1} \qquad \text{(Eq. 17-23)}$$

In the inset of Figure 17-12 the relationships between I_1, O_1, and O_2 make similar triangles, so that:

$$\frac{O_2 - O_1}{\Delta t} = \frac{I_1 - O_1}{K} \qquad \text{(Eq. 17-24)}$$

where K is considered the reach travel time for a selected steady flow discharge of a water particle through the given reach. From Equation 17-24 it follows that:

$$\frac{\Delta t}{K} = \frac{O_2 - O_1}{I_1 - O_1} \qquad \text{(Eq. 17-25)}$$

Combining Equations 17-23 and 17-25 gives:

$$C = \frac{\Delta t}{K} \qquad \text{(Eq. 17-26)}$$

[1/] Enough of the theory of convex sets for the purposes of this chapter is given in pages 41-42 of "An Introduction to Linear Programming," by A. Charnes, W. W. Cooper, and A. Henderson; John Wiley and Sons, Inc., New York, 1953.

from which comes the equation that defines Δt, the wave travel time and also the required routing interval:

$$\Delta t = C\ K \qquad \text{(Eq. 17-27)}$$

Discussion

This much of the theory is enough for making a workable routing method. The emphasis in this chapter is on working examples, not on theory, therefore the additional results from the theory are summarized in the next section without giving derivations or proofs. Further work can be done on some aspects of the Convex routing method but even in its present state the method is highly useful for most types of problems of routing flood flows through stream channels.

The theory as given so far can be used for exploratory routings by assuming magnitudes for any two of the variables in Equation 17-27 computing the third, and using Equation 17-21 with various inflow hydrographs. Such routings show the features of the Convex method. In Figure 17-12, for example, note that outflow begins at one routing interval, Δt, after inflow begins, which is to be expected for a stream reach because it takes water waves time to travel through the reach. It is chiefly this characteristic that distinguishes the Convex method from channel methods based on the continuity equation. In Convex routing the peak rate of the outflow hydrograph does not fall on the recession limb of the inflow hydrograph, as in reservoir methods. But, as in all routing methods, the maximum storage in the reach is attained when outflow equals inflow (at point A in Figure 17-12). The maximum storage is represented by the area under the inflow hydrograph to the left of point A minus the area of the outflow hydrograph to the left of point A. Also note that inflow I_2 does not appear in the working equation though it does appear in equations for other channel methods. This feature makes the Convex method a forecasting method. For example, if the routing interval is one day, today's inflow and outflow are known and local inflow is known or negligible, then tomorrow's outflow can be predicted accurately without knowing tomorrow's inflow. The predictive feature is more important for large rivers than for small streams because the routing interval for reaches of such streams is usually short.

Some Useful Relationships and Procedures

Of the equations so far given, only Equations 17-21 and 17-27 are needed in practical applications of the Convex method. The first is the working equation and the second an auxiliary equation used once before a routing begins. Several other relationships and procedures also useful in applications follow.

Determination of K. - K is the reach travel time for a selected steady-flow discharge and can be computed using Equation 17-7 substituting K for T_t. Example 17-8 shows a preferred method for selecting the discharge. The K used in the Muskingum routing method (refs. 2 and 3) may also be used as the K for the Convex method.

Determination of C. - From Equations 17-17 and 17-26 the parameter or routing coefficient C can be derived as the ratio of two velocities: that is, $C = V/U$, where V is the steady-flow water velocity related to the reach travel time for steady flow discharge, K, and U is considered the wave velocity related to the travel time of the wave through the reach, Δt. For practical purposes C may be estimated from an empirical relationship between C and V shown in Figure 17-13. The dashed line in the Figure is represented by the equation:

$$C = \frac{V}{V + 1.7} \qquad \text{(Eq. 17-28)}$$

In some applications it is more convenient to use Equation 17-28 than Figure 17-13. The "x" used in the Muskingum routing method (refs. 2 and 3) may also be used to approximate C. The approximation is:

$$C \approx 2\,x \qquad \text{(Eq. 17-29)}$$

In the Muskingum procedure the x is sometimes determined only to the nearest tenth; if this is done then C is approximated to the nearest two tenths and accurate routing results should not be expected.

Determination of Δt. - If C and K are known, from Equation 17-27, there is only one permissible routing interval. This permissible interval may be an inconvenient magnitude because it is either an unwieldy fraction of an hour or does not fit the given hydrograph. In selecting a suitable routing interval keep in mind that too large an interval will not accurately define the inflow hydrograph and that too small an interval will needlessly increase the effort required for the routing. A generally suitable rule of thumb to follow is that the selected routing time interval, Δt^*, should be no greater than 1/5 of the time from the beginning of rise to the time of the peak discharge of the inflow hydrograph, or:

$$\Delta t^* \leq \frac{T_p}{5} \qquad \text{(Eq. 17-30)}$$

where T_p is the time to peak (Chapter 16). If the hydrograph has more than one peak the interval should be selected using the T_p for the shortest of the rise periods of the important peaks. It is important that an end-point of a time interval fall at or near the inflow peak time and any other large change in rate.

* *

Procedure for routing through any reach length.- The relationship of K, C, and Δt is valid for one and only one routing reach length for a given time interval and inflow hydrograph. If Δt is to be changed to Δt^* (desired routing time interval) it follows from Equation 17-27 that either (1) C or K must be changed (Method 1) or, (2) routing through a series of subreaches, L^*, (Equation 17-32) must be made until the sum of the travel time of the Δt's for each subreach, L^*, equal the desired travel time, Δt^*, for the total reach, L (Method 2). Selection of either method

depends on the manner of computation and the consistency of the answers desired. Method 1 may be used when rough approximations of the routing effect are desired and manual computation is used. Method 2 is used when consistency of the routing is important or a computer is used. Consistency, as used here, refers to the changes in the outflow hydrograph (T_p and q_p) caused by varying Δt^*. If there is little change in the hydrograph when Δt^* is changed the routing is considered consistent.

In Method 1, the reach length is fixed, hence, K is fixed (Equation 17-17) and C must be modified by the empirical relationship:

$$C^* = 1 - (1 - C)^{\left(\frac{\Delta t^* + .5\Delta t}{1.5\Delta t}\right)} \qquad \text{(Eq. 17-31)}$$

where C^* is the modified routing coefficient required for use with Δt^*, C is the coefficient determined from Figure 17-13 or computed by Equation 17-28, Δt^* is the desired routing interval, and Δt is the routing interval determined from Equation 17-27. After selecting Δt^* the coefficient C^* is found by using either Equation 17-31 or Figure 17-14 (ES-1025 rev.)

Method 2 assumes that C and the desired routing interval Δt^* are fixed and the routing is made for a reach length L^*. From Equation 17-27, the desired travel time is:

$$K^* = \frac{\Delta t^*}{C} \qquad \text{(Eq. 17-32)}$$

From Equation 17-17 the proper routing reach length to match C and Δt^* is then:

$$L^* = (3600)(V)(K^*) \qquad \text{(Eq. 17-33)}$$

If L^* is less than the given reach length, L, the inflow hydrograph is repetitively routed until the difference between the sum of the L^*'s and L becomes less than the next L^*. The last routing in the reach is a fractional routing using C^* computed by Equation 17-31. The Δt used in Equation 17-31 is the time interval for routing through the fractional length increment of L, L^{**}. (See Example 17-11 Method 2).

If L^* is greater than the given reach length, L, the inflow hydrograph is routed once using Method 1. Example 17-11 illustrates the use of Methods 1 and 2.

Variability of routing parameters; selection of velocity, V.
As shown by preceding relationships, the magnitudes of the routing parameters C and K (and therefore of Δt) depend on the magnitude of the velocity V. For steady flow in natural streams this velocity varies with stage but the variation is not the same for all seasons of a year or for all reaches of a stream, nor does the velocity consistently increase or decrease with stage. For unsteady flow, velocity varies not only with stage but also with the rate of change of the stream flow.

These facts would appear to require a change in routing parameters for each operational step in a routing. But exploratory routings with the Convex method show that constant parameters must be used to conserve mass, that is, to make total outflow equal total inflow. The necessity for the use of constant parameters is a characteristic of coefficient routing equations, including not only Equation 17-21 but also with the Muskingum routing equation (refs. 2 and 3) and the Storage-Indication equations. Therefore all of the examples in this part show a use of constant parameters. In practice the parameters need not be constant for all steps of a routing but the more often they are changed the more likely that the total outflow will not equal total inflow.

The average, dominant, and peak velocities of one inflow hydrograph will nearly always differ from the corresponding velocities of another hydrograph. Even though a single value of V is used to get the constant values of C, K, and Δt for a routing, this V will nearly always be different for different inflow hydrographs to a reach. Each inflow hydrograph will need its own routing parameters determined from its own selected velocity. There are various methods of selecting the velocity.

One method, useful when a computer is used, computes the velocity as the average of velocities for all given discharges of the inflow hydrograph ≥ 50 percent of the peak discharge.

A manual method with the same objective as the machine method will be used in this chapter to make manual routings comparable to machine routings. In this method the dominant velocity of the inflow hydrograph is used to determine the parameters to be used in the routing. If the inflow hydrograph has a single peak the velocity is for a discharge equal to 3/4 of the peak inflow rate. If the inflow hydrograph has two or more peaks the velocity is for the discharge with the largest value of Tq, where:

$$Tq = (3/4\text{-discharge}) \times (\text{duration of } 3/4\text{-discharge}) \quad \text{(Eq. 17-34)}$$

The use of Equation 17-34 is illustrated in Example 17-8. Some additional remarks concerning the selected velocity are given in the paragraph preceding Example 17-7.

Examples.- The Convex method is generally used for routing hydrographs through stream reaches. It can also be used, without any change in procedure for routing mass curves through reaches. Examples of both uses will be given. The method can be used for routing through reservoirs but for this it is not as efficient as the mass-curve method of Example 17-1; therefore no examples of reservoir routing are given in this part. Examples are given showing various aspects of Convex routing.

Example 17-7 - Basic routine using assumed parameters.

Example 17-8 - Routing with parameters determined from reach data and with local inflow added at bottom of reach.

Example 17-9 - "Reverse Routing" or determining the inflow hydrograph for a given outflow hydrograph.

Example 17-10 - Routing of Mass Curve and method of getting the outflow hydrograph.

Example 17-11 - Routing any hydrograph through any reach. Method 1 and Method 2 are compared.

For the following examples it is assumed that stage-discharge and stage-end-area curves are available for the routing reach. These curves are used for determining the velocity, V, after the dominant discharge of the inflow hydrograph is obtained. In preliminary work such curves may not be available, in which case the velocity can be estimated during a field trip to the stream area, or a suitable velocity assumed, and the routing made as a tentative study; such routings need verification by routings based on reach data before making firm decisions about a project.

In the first example the values of C and Δt are assumed; therefore the reach length and K do not directly enter into the work:

Example 17-7.--Route the triangular inflow hydrograph of Figure 17-15 by the Convex computational method. Use assumed values of C = 0.4 and Δt = 0.3 hours. There is no local inflow into the reach.

1. Prepare the operations table.
Suitable headings and arrangement are shown for the first three columns in Table 17-14. The "remarks" column is used here to explain the steps; it is not needed in routine work.

2. Tabulate the inflow rates at accumulated times, using intervals of Δt.
The accumulated times at intervals of Δt = 0.3 hours are shown in column 1 of Table 17-14. The inflow rates at these times are taken from the inflow hydrograph of Figure 17-15 and shown in column 2.

3. Prepare the working equation.
Since C = 0.4 then (1 - C) = 0.6 and the working equation is $O_2 = (1 - C)\ O_1 + C\ I_1 = 0.6\ O_1 + 0.4\ I_1$. When inflow ceases the working equation is $O_2 = 0.6\ O_1$.

4. Do the routing.
Follow the steps shown in the remarks column of Table 17-14.

The computational work in step 4 can usually be done on most desk-calculators by using a system of making the two multiplications and the addition in one machine operation.

The outflow hydrograph of Table 17-14 is plotted on Figure 17-15. The circled points are the outflow discharges obtained in the routing. Discharges between the points are found by connecting the points with a smooth curve. Sometimes the routing points do not define the peak region

Table 17-14 Basic operations in the Convex routing method.

Time (hrs)	Inflow, I (cfs)	Outflow, O (cfs)	Remarks
(1)	(2)	(3)	
0	0	0	Given.
.3	800	0 1/	$O_2 = 0.6(0) + 0.4(0) =$ zero
.6	1600	320	$O_2 = 0.6(0) + 0.4(800) = 320$
.9	2400	832	$O_2 = 0.6(320) + 0.4(1600) = 832$
1.2	3200	1459	$O_2 = 0.6(832) + 0.4(2400) = 1459$
1.5	4000	2155	$O_2 = 0.6(1459) + 0.4(3200) = 2155$
1.8	3520	2893	$O_2 = 0.6(2155) + 0.4(4000) = 2893$
2.1	3040	3144	$O_2 = 0.6(2893) + 0.4(3520) = 3144$
2.4	2560	3102	$O_2 = 0.6(3144) + 0.4(3040) = 3102$
2.7	2080	2885	$O_2 = 0.6(3102) + 0.4(2560) = 2885$
3.0	1600	2563	$O_2 = 0.6(2885) + 0.4(2080) = 2563$
3.3	1120	2178	$O_2 = 0.6(2563) + 0.4(1600) = 2178$
3.6	640	1755	$O_2 = 0.6(2178) + 0.4(1120) = 1755$
3.9	160	1309	$O_2 = 0.6(1755) + 0.4(640) = 1309$
4.2	0 2/	849	$O_2 = 0.6(1309) + 0.4(160) = 849$
4.5	0	509	$O_2 = 0.6(849) = 509$ I_1 = zero.
4.8	0	305	$O_2 = 0.6(509) = 305$ " " "
5.2	0	183	$O_2 = 0.6(305) = 183$ " " "
5.5	0	110	$O_2 = 0.6(183) = 110$ " " "
etc.	etc.	etc.	etc.

1/ Outflow starts at $\Delta t = 0.3$ hrs.

2/ Inflow ceases at 4.0 hrs.

well enough; this usually happens when the routing interval is large. In such cases the peak is estimated by use of a smooth curve or the routing is repeated using smaller intervals (see Example 17-11 for use of Δt^*).

The recession curve or tail of the outflow hydrograph continues to infinity, the discharges getting smaller with every step but never becoming zero. This is a characteristic of most routing methods. In practice the recession curve is either arbitrarily brought to zero at some convenient low discharge or the routing is stopped at some low discharge as shown in Figure 17-15.

The next example is typical of the routine used in practice. Routing parameters are obtained from reach data and local inflow is added in the conventional manner. Local inflow is the (usually) small flow from the contributing area between the head and foot of a reach. Local inflow and the inflow into the head of the reach together make up the total flow from the drainage area above the foot of the reach. The local inflow is generally given as a hydrograph made with reference to the foot of the routing reach. When it is added to the routed outflow the sum is the total outflow hydrograph.

Example 17-8.--The inflow hydrograph in Figure 17-16 is to be routed through a reach having a low-flow channel length of 14,900 feet and a valley length of 12,400 feet. Stage-discharge and stage-end-area curves for the reach are available (not illustrated). A hydrograph of local inflow is given in Figure 17-16. Obtain the total outflow hydrograph for the reach.

<u>1. Determine the discharge to be used for getting the velocity V</u>.
The inflow hydrograph has two peaks and it is not readily apparent which peak is the dominant one, therefore the rule expressed by Equation 17-34 will be used. The 3/4-discharge for the first peak is 3,750 cfs with a duration of 2.63 hours; for the second, 2,680 cfs with a duration of 5.35 hours. Then Tq = 3750(2.63) = 9,850 cfs-hrs for the first peak and Tq = 2680(5.35) = 14,320 cfs-hrs for the second, therefore the second discharge will be used.

<u>2. Determine the velocity, V</u>.
Enter the stage-discharge curve for the reach with the selected 3/4-discharge from step 1 and find the stage for that flow. Then enter the stage-end-area curve with that stage and get the end-area in square feet. The velocity is the discharge divided by the end area. For this example V will be taken as 3.0 fps.

<u>3. Determine K</u>.
The reach has two lengths, one for the low-flow channel, the other for the valley. From an examination of the stage-discharge curve and the inflow hydrograph it is evident that most of the flow will exceed the capacity of the low-flow channel, therefore use the valley length. This is given as 12,400 feet. By Equation 17-17, using $T_t = K$, the value of K = 12400/3600(3.0) = 1.15 hours by a slide-rule computation.

4. Determine C.
Enter Figure 17-13 with V = 3.0 fps and find C = 0.65.

5. Compute Δt.
Using results from steps 3 and 4, and by Equation 17-27, $\Delta t = 0.65 (1.15) = 0.745$ hours. Round to 0.75 hours.

6. Prepare an operations table for the routing.
Suitable headings and arrangement are shown in Table 17-15.

7. Tabulate accumulated time at intervals of Δt and the discharges for inflow and local inflow at those times.
The times are given in column 1 of Table 17-15, inflows in column 2, and local inflows in column 4. Inflows and local inflows are taken from the given hydrographs, which are shown in Figure 17-16.

8. Prepare the working equation.
From step 4, C = 0.65 so that (1 - C) = 0.35. The working equation is $O_2 = 0.35\ O_1 + 0.65\ I_1$.

9. Do the routing.
Follow the routine used in Table 17-14 to get the outflows for column 3 of Table 17-15.

10. Get the total outflow hydrograph.
Add the local inflows of column 4, Table 17-15, to the routed outflows of column 3 to get the total outflows for column 5. This step completes the example. The total outflow hydrograph is shown in Figure 17-16.

Note in Figure 17-16 that the routed outflow peaks are not much smaller than the inflow peaks. The first routed outflow peak is 93.0 percent of its respective inflow peak, and the second 97.7 percent of its inflow peak. The reach has relatively small storage when compared with the inflow volumes; the first inflow peak has less volume associated with it than the second and it is reduced more than the second.

The next example illustrates a routine sometimes needed to get the upstream hydrograph when the downstream one is given. The working equation for this routine is a rearranged form of Equation 17-21:

$$I_1 = \frac{1}{C} O_2 - \frac{(1 - C)}{C} O_1 \qquad \text{(Eq. 17-35)}$$

Example 17-9.--Obtain the inflow hydrograph of a reach from the total outflow hydrograph by use of reverse routing. The total outflow hydrograph and local inflow are given in Table 17-16.

1. Determine the routing coefficient C and the routing interval Δt.
Follow the procedure of steps 1 through 5 of Example 17-8. For this example C = 0.44 and Δt = 0.5 hrs.

Table 17-15 Operations table for Example 17-8.

Time (hrs.)	Inflow (cfs)	Outflow (cfs)	Local Inflow (cfs)	Total Outflow (cfs)
(1)	(2)	(3)	(4)	(5)
0	0	0	0	0
.75	380	0 1/	110	110
1.50	1400	247	430	677
2.25	3000	996	830	1826
3.00	4450	2299	1000	3299
3.75	5000	3697	890	4587
4.50	4600	4544	650	5194
5.25	3750	4580	460	5040
6.00	2800	4040	320	4360
6.75	2100	3234	2200	3454
7.50	1600	2497	180	2677
8.25	1280	1914	170	2084
9.00	1150	1502	210	1712
9.75	1210	1273	310	1583
10.50	1480	1232	470	1702
11.25	1880	1393	650	2043
12.00	2360	1710	830	2540
12.75	2880	2132	950	3082
13.50	3250	2618	1000	3618
14.25	3500	3029	970	3999
15.00	3580	3335	880	4215
15.75	3480	3494	780	4274
16.50	3240	3485	650	4135
17.25	2930	3326	550	3876
18.00	2600	3069	470	3539
18.75	2280	2764	400	3164
19.50	1980	2449	330	2779
20.25	1730	2144	280	2424
21.00	1480	1875	230	2105
21.75	1280	1618	190	1808
27.50	1130	1398	150	1548
23.25	980	1224	120	1344
24.00	850	1065	100	1165
24.75	720	925	90	1015
25.50	620	792	80	872
26.25	530	680	70	750
27.00	450	582	60	642
27.75	400	496	50	546
28.50	350	434	40	474
29.25	310	353	30	383
30.00	270	325	20	345
etc.	etc.	etc.	etc.	etc.

1/ Outflow starts at $\Delta t = 0.75$ hrs.

2. Prepare the operations table for the routing.
Suitable headings and arrangements are shown in Table 17-16.

3. Tabulate accumulated time at intervals of Δt and the discharges for total outflow and local inflow at those times.
The times are given in column 1 of Table 17-16, total outflows in column 2, and local inflows in column 3. The total outflow (but not the local inflow) is shown in Figure 17-17.

4. Determine the outflows to be routed upstream.
A value in column 2, Table 17-16, minus the corresponding value in column 3 gives the outflow for column 4, which contains the outflows to be routed upstream.

5. Prepare the working equation.
C is given in step 1 as 0.44. By Equation 17-35, $I_1 = 2.27\ O_2 - 1.27\ O_1$.

6. Do the routing.
The routine is slightly different from that in Table 17-14. Using values from Table 17-16, the sequence is: for outflow time 0.5 hrs, $I_1 = 2.27(0) - 1.27(0) = 0$, which is recorded for inflow time zero; at outflow time 1.0 hrs, $I_1 = 2.27(163) - 1.27(0) = 370$, recorded for inflow time 0.5 hrs; for outflow 1.5, $I_1 = 2.27(478) - 1.27(163) = 878$, recorded for inflow time 1.0 hrs; and so on. The work is easily done by accumulative positive and negative multiplication on a desk calculator. The inflow hydrograph to time 7.5 hours is plotted on Figure 17-17.

It will sometimes happen in reverse routing that the working equation gives negative values for the inflow. This occurs when the total outflow hydrograph or the local inflow is in error.

The next example shows the downstream routing of a mass curve of inflow. The routine is the same as that for Example 17-7. The outflow hydrograph can be obtained from the mass outflow curve by a series of simple calculations; these outflows must be plotted at midpoints of time increments, not at end points.

Example 17-10.--Route the mass curve of inflow of Figure 17-18 by the Convex method. There is no local inflow.

1. Determine the routing coefficient C and the routing interval Δt.
Follow the procedure of steps 1 through 5 of Example 17-8. For this example $C = 0.40$ and $\Delta t = 0.3$ hrs.

2. Prepare the operations table for the routing.
Suitable headings and arrangement are shown in Table 17-17.

Table 17-16 Operations table for Example 17-9

Time (hrs)	Total Outflow (cfs)	Local Inflow (cfs)	Outflow to be routed (cfs)	Inflow (cfs)
(1)	(2)	(3)	(4)	(5)
0	0	0	0	0
.5	120	120	0 1/	370
1.0	310	147	163	878
1.5	680	202	478	1508
2.0	1250	318	932	2278
2.5	1850	325	1525	2978
3.0	2490	325	2165	3398
3.5	3030	322	2708	3648
4.0	3440	318	3122	3793
4.5	3700	280	3420	3899
5.0	3900	269	3631	3819
5.5	3940	226	3714	3539
6.0	3840	203	3637	2972
6.5	3500	156	3344	2370
7.0	3000	85	2915	1800
7.5	2485	61	2424	1300
8.0	1960	31	1929	etc.
etc.	etc.	etc.	etc.	

1/ Outflow starts at Δt = 0.5 hours.

3. Tabulate accumulated time at intervals of Δt and the mass inflows at those times.
The times are given in column 1 of Table 17-17 and mass inflows in column 2.

4. Prepare the working equation.
C is given in step 1 as 0.40. By Equation 17-21, $O_2 = 0.6\ O_1 + 0.4\ I_1$.

5. Do the routing.
The routine is exactly the same as that in Table 17-14. For example, at inflow time 2.7 hrs, O_2 is computed using inflow and outflow for the previous time or $O_2 = 0.6(3707) + 0.4(5952) = 4605$ cfs-hrs.

(Note: If only the mass outflow is needed the work stops with step 5. If the outflow hydrograph is also needed, the following steps are also taken.)

6. Compute increments of outflow.
These are the differences shown in column 4, Table 17-17.

7. Compute average rates of outflow.
Dividing the increment of outflow of column 4, Table 17-17, by the increment of time (in this case, 0.3 hrs) gives the average rate of outflow for the time increment. For example, between 1.8 and 2.1 hours in Table 17-17, the time increment is 0.3 hrs and the outflow increment is 906 cfs-hrs; then the average rate is 906/0.3 = 3,020 cfs. The average rates must be plotted as midpoints between the two accumulated times involved; for this case, 3020 cfs is plotted at a time of (1.8 + 2.1)/2 = 1.95 hours.

The mass inflow, mass outflow, and rate hydrograph are plotted in Figure 17-18.

The next example shows how to route any hydrograph through any reach length. Methods 1 and 2 are compared.

Example 17-11.--Route the inflow hydrograph of Figure 17-19 through a reach 30,000 feet long. Assume no local inflow.

Method 1

1. Determine desired routing time interval, Δt^*.
Following the rule expressed in Equation 17-30, Δt^* will be 0.4 hrs.

2. Determine routing coefficient, C, and routing interval Δt.
If a stage-discharge-velocity table for a typical section in the reach is used, the average velocity V is determined using the method from page 17-54, and C is computed using Equation 17-28. If a rating table is not used the C or V must be assumed; in this case, let C = 0.72. Rearranging Eq. 17-28 gives $V = 1.7C/(1-C) =$

Table 17-17 Operations table for Example 17-10.

Time (hrs.)	Mass Inflow (cfs-hrs)	Mass Outflow (cfs-hrs)	Increment of Outflow (cfs-hrs)	Outflow Rate (cfs)
(1)	(2)	(3)	(4)	(5)
0	0	0	0	0
.3	120	0 1/	48	160
.6	480	48	173	577
.9	1080	221	344	1146
1.2	1920	565	542	1806
1.5	3000	1107	757	2523
1.8	4128	1864	906	3020
2.1	5112	2770	937	3123
2.4	5952	3707	898	2993
2.7	6648	4605	817	2723
3.0	7200	5422	711	2370
3.3	7608	6133	590	1966
3.6	7872	6723	460	1533
4.2	7992	7183	324	1080
4.5	7992	7507	194	647
4.8	7992	7701	116	387
5.2	7992	7817	70	233
5.5	7992	7887	etc.	etc.
etc.	etc.	etc.		

1/ Outflow starts at Δt = 0.3 hours

$(1.7)(.72)/0.28 = 4.37$ fps. Combining Equations 17-17 and 17-27,

$$K = \frac{L}{3600V} = \frac{\Delta t}{C} \text{ or } \Delta t = \frac{CL}{3600V} = (.72)(30000)/(3600)(4.37) =$$

1.37 hrs. Use 1.4 hrs. Δt is also the wave travel time through the entire reach.

3. Determine C*
Using Equation 17-31 with $\Delta t = 1.4$ hrs, $\Delta t^* = 0.4$ hrs, and $C = 0.72$

$$C^* = 1 - (1-.72)^{\left(\frac{0.4+0.5(1.4)}{1.5(1.4)}\right)} = 1-(.28)^{\left(\frac{1.1}{2.1}\right)} = 1-(.28)^{0.524} =$$

$1-0.51 = 0.49$.

4. Prepare an operations table for the routing.
Suitable headings and arrangement are shown in Table 17-18.

5. Tabulate accumulated time intervals of Δt^* and the inflow discharges for those times.
The times are given in column 1 Table 17-18, the inflows taken from Figure 17-19 in column 2.

6. Prepare the working equation.
From step 3, $C^* = 0.49$. Using Equation 17-21 $O_2 = (1 - C^*)\,O_1 + C^*I_1$ or $O_2 = 0.51\,O_1 + 0.49\,I_1$. Solutions of this equation can easily be made by accumulative multiplication or a desk calculator.

7. Do the routing.
Follow the routine of Table 17-14. The outflows are shown in column 3 of Table 17-18.

8. Determine the times for the outflow.
Outflow begins at the end of the first Δt (not Δt^*) interval.

With $\Delta t = 1.4$ hrs, show this time in column 4 of Table 17-18 in the row where outflow begins. Get succeeding times by adding Δt^* intervals, 0.4 hours in this case, as shown in column 4. In plotting or otherwise displaying the inflow and outflow hydrographs they are put in their proper time order, using columns 1 and 4, as shown in figure 17-19.

Method 2

1. Determine desired routing time interval, Δt^*.
Same as Method 1, $\Delta t^* = 0.4$ hr.

2. Determine routing coefficient C.
The routing coefficient "C" for each subreach is computed from the outflow hydrograph of the preceding subreach as done in Step 2, Method 1. A constant C may be used for the entire reach but the resultant hydrograph will vary from one produced by recomputing C for each subreach. For simplicity in this example, a constant $C = 0.72$ is assumed. $V = 4.37$ fps.

Table 17-18 Operations table for Example 17-11 Method 1.

Time Inflow (hrs)	Inflow (cfs)	Outflow (cfs)	Time Outflow (hrs)
(1)	(2)	(3)	(4)
0	0	0	
.4	260	0 1/	1.4
.8	980	127	1.8
1.2	2100	545	2.2
1.6	3120	1307	2.6
2.0	3500	2195	3.0
2.4	3220	2834	3.4
2.8	2630	3023	3.8
3.2	1960	2830	4.2
3.6	1470	2404	4.6
4.0	1120	1946	5.0
4.4	840	1541	5.4
4.8	630	1198	5.8
5.2	455	920	6.2
5.6	345	692	6.6
6.0	265	522	7.0
6.4	180	396	7.4
6.8	130	290	7.8
7.2	100	212	8.2
7.6	75	157	8.6
8.0	60	117	9.0
8.4	45	89	9.4
8.8	35	67	9.8
9.2	20	51	10.2
9.6	10	36	10.6
10.0	0	23	11.0
etc.	etc.	12	11.4
		6	11.8
		3	12.2
		2	12.6
		1	13.0
		etc.	etc.

1/ Outflow starts at Δt = 1.4 hours

3. Determine length of subreach L^*.
This is the length of reach required to satisfy the relationship of Equation 17-26 with $C = 0.72$ and $\Delta t^* = 0.4$ hrs. Combining Equations 17-26 and 17-17 (let $K = T_t$) we have $L^* = (\Delta t)(V)(3600)/C = (0.4)(4.37)(3600)/0.72 = 8740$ ft.

4. Compare the total of subreach lengths, ΣL^* with the given reach length, L.
For $\Sigma L^* \leq L$ go to step 5
For $\Sigma L^* > L$ go to step 7

In this example $\Sigma L^*_{n=1} = 8740$

$\Sigma L^*_{n=2} = 17480$

$\Sigma L^*_{n=3} = 26220$

$\Sigma L^*_{n=4} = 34960$

Therefore, the first three routings are made by going to step 5 and the last routing by going to step 7.

5. Prepare working equation and do the routing.
Using Equation 17-21 and the routing coefficient computed in step 2, $O_2 = (1 - C)O_1 + CI_1 = 0.28\ O_1 + 0.72 I_1$. The outflows for each subreach are shown in Table 17-19.

6. Go to step 2.

7. Determine the length of the remaining subreach to be routed.
Subtract the ΣL^* of the 3 completed routings, i.e., 26220 ft from the total reach length to get the remaining reach length to be routed. $L^{**} = 30000 - 26220 = 3780$ ft.

8. Determine the Δt time interval for the remaining subreach.
The time interval used here is the same as the wave travel time through the remaining subreach. Combining Equations 17-17 and 17-27 as in step 2 Method 1 $\Delta t^{**} = \frac{CL^{**}}{3600V} = (0.72)(3780)/(3600)(4.37) = 0.173$ hrs.

9. Determine the modified routing coefficient C^*.
Using Equation 17-31 with $\Delta t^{**} = 0.173$, $\Delta t^* = 0.4$ and $C = 0.72$,

$$C^* = 1-(1-C)^{\left(\frac{\Delta t^* 0.5 \Delta t^{**}}{1.5 \Delta t^{**}}\right)} = 1-(1-.72)^{\left(\frac{0.4+0.5(.173)}{1.5\ (.173)}\right)} =$$

$$1-(.28)^{\left(\frac{.4865}{.2595}\right)}\quad 1-(.28)^{1.89} = 1 - .090 = .91$$

10. Prepare working equation.
Following Method 1 $O_2 = (1-C^*)O_1 + C^* I_1 = (1 - .91)\ O_1 + 0.91 I_1 = .09\ O_1 + 0.91\ I_1$.

11. Do the routing.
The outflow for the fractional routing are shown in column 6 Table 17-19.

12. Determine the time for the routing.
The hydrograph for each subreach routing is set back one Δt time interval. In this example the first three routings are set back 0.4 hrs each and the last (fractional) routing is set back 0.173 hrs (round to 0.2 hrs). See column 7 Table 17-19 and Figure 17-19.

When C* and Δt* are used and local inflow is to be added, the local inflow must be used in its actual time position regardless of Δt and Δt*. That is, the local inflow is not shifted back or forth because it is not affected by the use of C* and Δt*.

*   *

Effects of transmission losses on routed flows

A flood hydrograph is altered by transmission losses occurring during passage of the flow through a reach. The amount of loss depends on the percolation rate of the channel, the wetted perimeter of channel during flow, and the duration of flow for a particular wetted perimeter (Chapter 19). Transmission loss varies with the amount of flow in the channel which means that the most accurate method of deducting the transmission loss from the routed flow will be on an incremental flow basis. It is seldom worthwhile to handle it in this manner unless the transmission loss is very large.

An acceptable practice for handling transmission losses is to route the inflow hydrograph in the usual manner and afterwards deduct a suitable quantity of flow from the outflow hydrograph (mainly from the rising limb). If that outflow is to be routed downstream again, the manner of flow deduction will not be critical. In some cases it may be reasonable to assume that local inflow will be completely absorbed by transmission losses, thus no local inflow is added to the unmodified outflow hydrograph. In other cases local rainfall may completely satisfy transmission losses, requiring unmodified local inflow to be added to the unmodified outflow hydrograph. The use of detailed procedures outlined in Chapter 19, "Transmission Losses", may be necessary for more complex situations.

Routing through a system of channels

The methods of channel routing given in Examples 17-7 through 17-11 are used for individual reaches of a stream. Ordinarily a routing progresses from reach through reach until the stages, rates, or amounts of flow are known for selected points in the entire stream system of a watershed. The method of progression will be illustrated using a schematic diagram or "tree graph" of a stream system. A typical graph is given in Figure 17-20. It does not need to be drawn to scale. The main purpose of the graph is to show the reaches in their proper relationship to each other, but various kinds of data can be written down at their respective points of application to make the graph a complete reference during the routing.

Routing through a stream system begins at the head of the uppermost reach. If there is more than one possible starting place, as in Figure 17-20, the most convenient should be chosen.

Table 17-19 Operation table for Example 17-11 Method 2

Time Inflow (hrs)	Inflow (cfs)	Outflow ΣL*=8740 (cfs)	Outflow ΣL*=17480 (cfs)	Outflow ΣL*=26220 (cfs)	Outflow ΣL*=30000 (cfs)	Outflow Time (hrs)
(1)	(2)	(3)	(4)	(5)	(6)	(7)
0	0					
.4	260	0 1/				
.8	980	187	0			
1.2	2100	758	135	0		
1.6	3120	1724	584	97	0 2/	1.4
2.0	3500	2729	1405	447	88	1.8
2.4	3220	3284	2358	1137	415	2.2
2.8	2630	3238	3025	2016	1072	2.6
3.2	1960	2800	3178	2743	1931	3.0
3.6	1470	2195	2906	3056	2670	3.4
4.0	1120	1673	2394	2948	3021	3.8
4.4	840	1275	1875	2549	2955	4.2
4.8	630	962	1442	2064	2586	4.6
5.2	455	723	1096	1617	2111	5.0
5.6	345	530	827	1242	1661	5.4
6.0	265	397	613	944	1280	5.8
6.4	180	302	457	706	974	6.2
6.8	130	214	345	527	730	6.6
7.2	100	154	251	396	545	7.0
7.6	75	115	181	292	409	7.4
8.0	60	86	133	212	303	7.8
8.4	45	67	99	155	220	8.2
8.8	35	51	76	115	161	8.6
9.2	20	40	58	87	119	9.0
9.6	10	25	45	66	90	9.4
10.0	0	14	31	51	68	9.8
etc.	etc.	4	19	36	53	10.2
		1	8	24	38	10.6
		0	3	13	25	10.0
		etc.	1	6	14	11.4
			0	2	7	11.8
			etc.	1	2	12.2
				0	1	12.6
				etc.	0	13.0
					etc.	

1/ Outflow from subreach 1, 2, & 3 starts Δt* = 0.4 hours after inflow starts into each subreach.

2/ Outflow from subreach 4 starts Δt = 0.2 hours (rounded from 0.17 hours) after inflow starts into subreach 4.

The first major step in routing through a stream system is to develop the routing parameters, C and Δt, for each reach. Many times it is necessary to use Δt^* to make the routing interval uniform through the stream system; these parameters should be obtained before the routing begins. The method of developing the parameters C, K, and Δt is given in steps 1 through 5 of Example 17-8. The method of determining C* and Δt^* is given in steps 1 through 3 of Example 17-11.

The second major step is the development of the inflow hydrographs at heads of uppermost reaches and of local inflow hydrographs for all reaches. The methods of Chapter 16 are used.

The third major step is the routing. For routing any particular flood on the stream system pictured in Figure 17-23 a suitable sequence is as follows:

1. Route the inflow hydrograph at (a) through reach (a,b).

2. Add local inflow of reach (a,b) to the routed outflow to get the total outflow hydrograph, which becomes the inflow hydrograph for reach (b,c). It should be noted here that local inflow for a reach is usually added at the foot of the reach. These may be circumstances, however, in which the local inflow should be added at the beginning of the reach. The proper sequence for adding local inflow can be determined only by evaluating each reach.

3. Route the total outflow from reach (a,b) through reach (b,c).

4. Add local inflow of reach (b,c) to the routed outflow to get the total outflow hydrograph for that tributary.

5. Route the inflow hydrograph at (d) through reach (d,c).

6. Add local inflow of reach (d,c) to the routed outflow to get the total outflow hydrograph at point (c).

7. Add the total outflow hydrographs from reaches (b,c) and (d,c), steps 4 and 6, to get the total outflow hydrograph at point (c).

8. Route the total hydrograph at point (c) through reach (c,f).

9. Add local inflow of reach (c,f) to the routed outflow to get the total outflow hydrograph at point (f).

10. Route the inflow hydrograph at point (e) through reach (e,f).

11. Add local inflow of reach (e,f) to the routed outflow to get the total outflow hydrograph for that tributary.

12. Route the inflow hydrograph at point (g) through reach (g,f).

13. Add local inflow of reach (g,f) to the routed outflow to get the total outflow for that tributary at point (f).

14. Add the total outflow hydrographs from reaches (c,f), (e,f), and (g,f), steps 9, 11, and 13 to get the total outflow hydrograph at point (f).

15. Route the total hydrograph at point (f) through reach (f,h).

16. Add local inflow of reach (f,h) to the routed outflow to get the total outflow hydrograph for reach (f,h).

17. Route the total hydrograph at point (h) through reach (h,i).

18. Add local inflow of reach (h,i) to the routed outflow to get the total outflow hydrograph for reach (h,i).

19. Route the inflow hydrograph at point (j) through reach (j,k).

20. Add local inflow of reach (j,k) to the routed outflow to get the total outflow hydrograph for reach (j,k).

21. Route the hydrograph at point (k) through reach (k,i).

22. Add local inflow of reach (k,i) to the routed outflow to get the total outflow hydrograph for this tributary.

23. Add the total outflow hydrographs from reaches (h,i) and (k,i), steps 18 and 22, to get the total outflow hydrograph for point (i).

24. Route the hydrograph at point (i) through reach (i,ℓ).

25. Add local inflow of reach (i,ℓ) to the routed outflow to get the total outflow hydrograph at point (ℓ). This completes the routing for a particular flood on this stream system.

When manual computations are used, an operations table with times, inflow hydrographs and local inflows tabulated in their proper sequence is useful. Blank columns are left for the routed outflows and total outflows, which are tabulated as routing progresses. Above the appropriate columns the required data and routing parameters are tabulated so that the table becomes a complete reference for the routing. A sample operations table for routing by Method 2 is shown as Table 17-20. After the inflow hydrograph and local inflows are tabulated the sequence of the work is as follows:

Tabulate the reach numbers in the order in which the routing will progress; perform the routings as shown in Example 17-11 and continue in this manner through the stream system. Note the routed outflow at 1.17 hrs which is rounded to 1.0 hrs. Theoretically, the outflow hydrograph should be interpolated on a multiple of Δt to properly position the hydrograph in relation to time. The linear interpolation equation is:

$$q_i = q_i + (q_{i+i} - q_i) \times \frac{\Delta t^* - \Delta t}{\Delta t^*} \qquad \text{(Eq. 17-36)}$$

where: q_i and q_{i+i} are consecutive discharges, Δt^* is the desired time interval and Δt is the required time interval of the partial routing. When using Method 2, Δt is always less than Δt^*.

If the interpolation step is omitted and the starting times rounded as in Table 17-20 it is recognized an error is introduced, the magnitude of which depends on the relative values of Δt and Δt^*.

* *

Table 17-20. Portion of a typical operations table for routing through a stream system.

REACH	16				15			14					
L(ft)	19300.				4000.			37000.					
V(fps)	2.9				4.8			3.2					
C	.63	.63			.74			.65	.65				
Δt*(hrs)	1.00	1.00			1.00			1.00	1.00				
L* (ft)	16541.	2759.			4000.			17486.	17486.	2028.			
Δt (hrs)		.17			.17					.12			
C*		.986			1.00					.998			
Time	Inflow	Outflow Subrch1 @16540.'	Outflow	Local Inflow	Total Outflow	Outflow	Local Inflow	Total Outflow	Outflow Subrch1 @17436.'	Outflow Subrch2 @34972.'	Outflow	Local Inflow	Total Outflow
hrs.	(All Discharges in cfs)												
0	0			0	0	0[2/]	0	0				80	80
1	450	0	0[1/]	190	190	190	40	230	0			80	80
2	1680	284	280	700	980	980	140	1020	150	0	0[3/]	80	80
3	3600	11[illegible]3	1151	1000	2151	2151	300	2451	715	97	97	600	697
4	5340	2698	2678	810	3488	3488	445	3933	1844	498	498	850	1248
5	6000	4363	4340	520	4860	4860	500	5360	3202	1371	1371	690	2061
6	5520	5394	5380	315	5695	5695	460	6155	4605	2560	2560	410	2970
7	4500	5473	5472	205	5677	5677	380	6057	5612	3887	3887	200	4087
8	3360	4860	4868	120	4988	4988	280	5268	5901	5007	5007	100	5107
9	2520	3915	3928	75	4003	4003	210	4213	5490	5588	5588	80	5668
10	1920	3036	3048	45	3093	3093	160	3253	4660	5525	5525	80	5605
11	1440	2333	2343	25	2368	2368	120	2488	3745	4963	4963	80	5043
12	1080	1770	1778	15	1793	1793	90	1883	2928	4173	4173	80	4253
13	780	1335	1341	10	1351	1351	65	1416	2249	3365	3365	80	3445
14	580	986	990	8	998	998	50	1048	1707	2640	2640	80	2720
15	450	735	739	5	744	794	35	779	1279	2034	2037	80	2117
etc.	etc.	etc.	etc.	etc.	etc.	etc.	etc.	etc.	etc.	etc.	etc.	etc.	etc.

1/ Outflow begins at 1.00 hrs., rounded from 1.17 hrs.
2/ Outflow begins at 0.00 hrs., rounded from 0.17 hrs.
3/ Outflow begins at 2.00 hrs., rounded from 2.12 hrs.

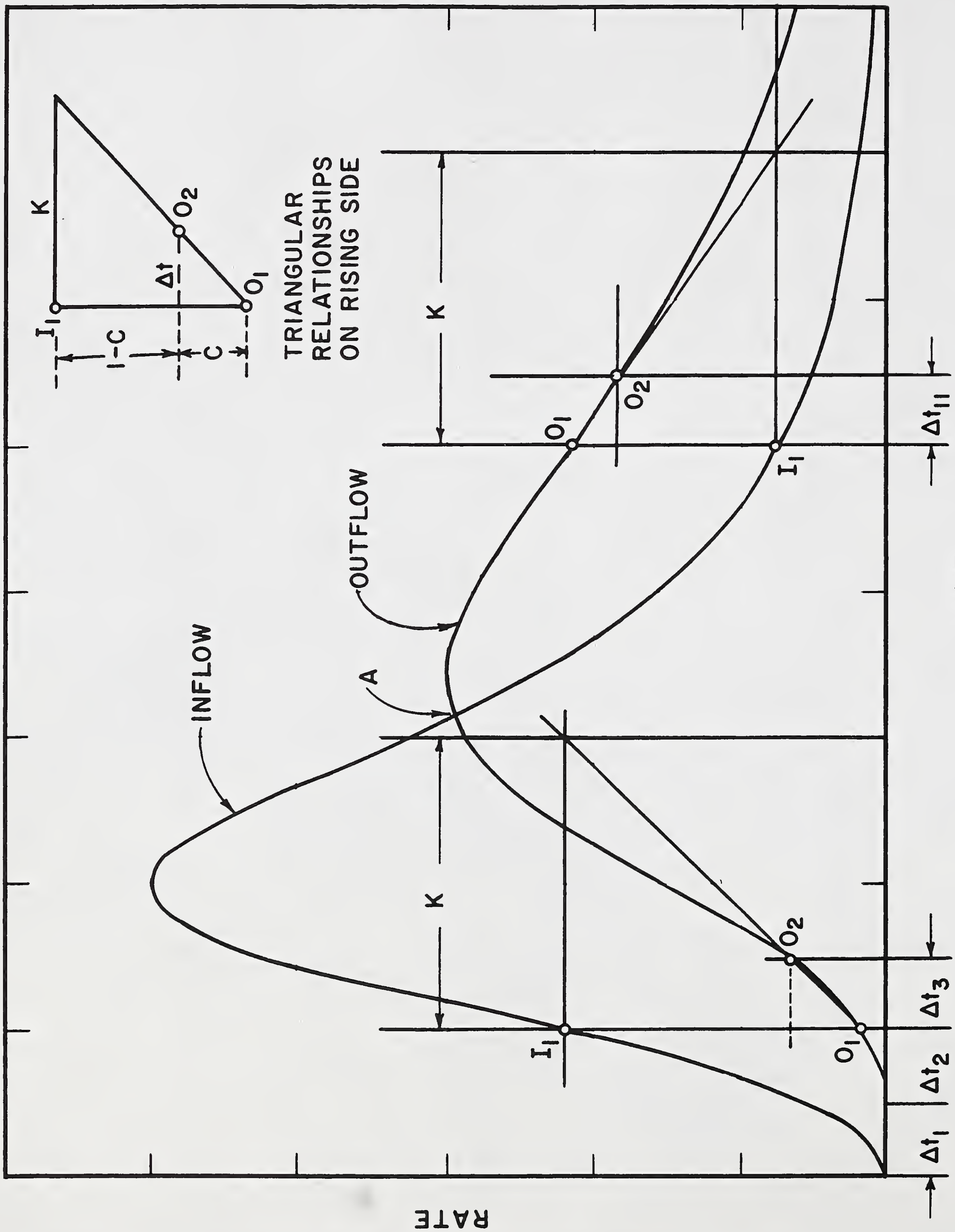


Figure 17-12. Relationships for Convex method of channel routing.

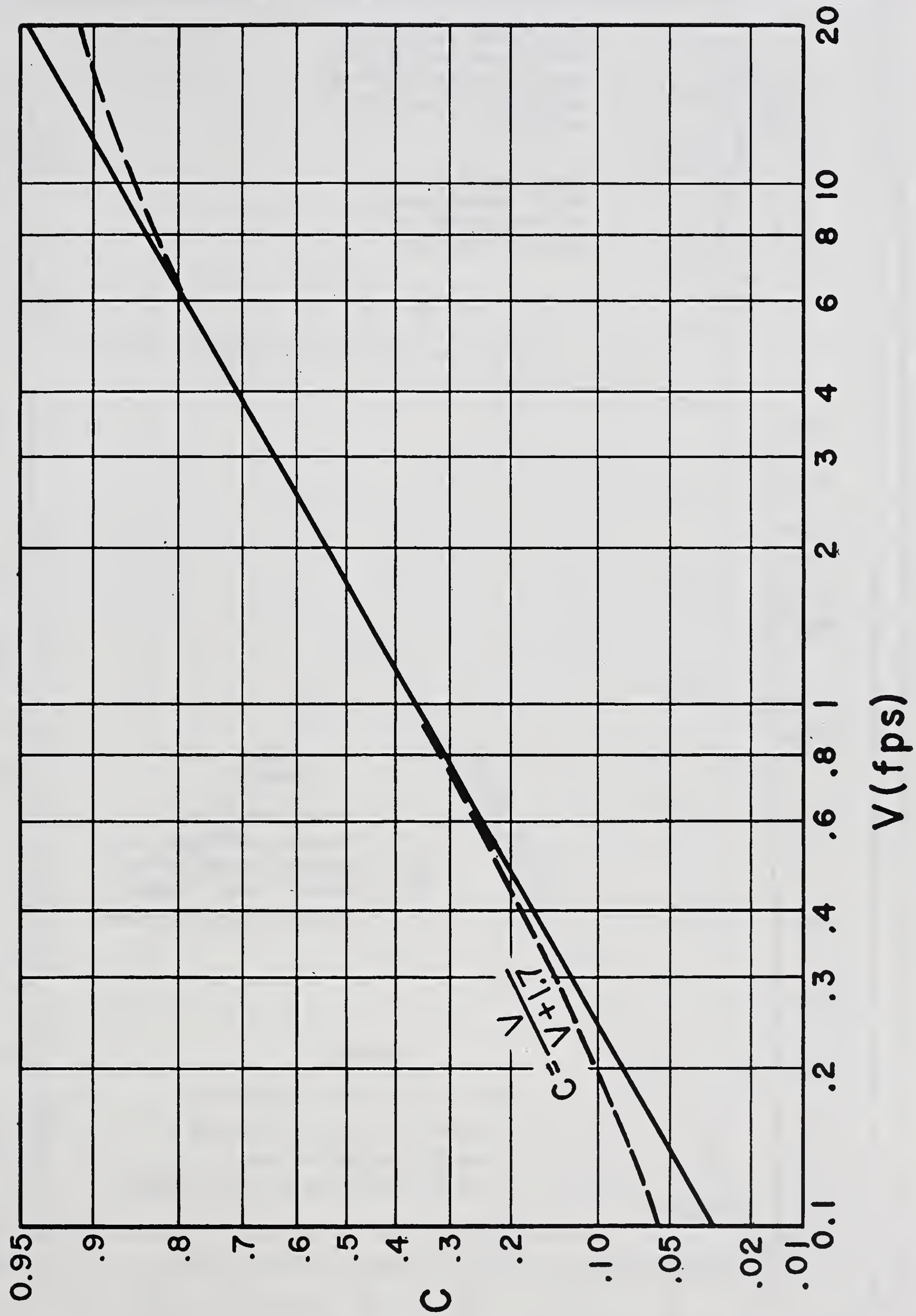
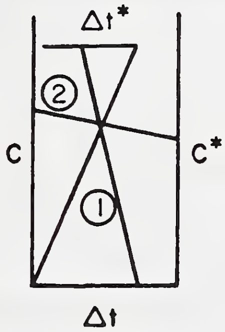


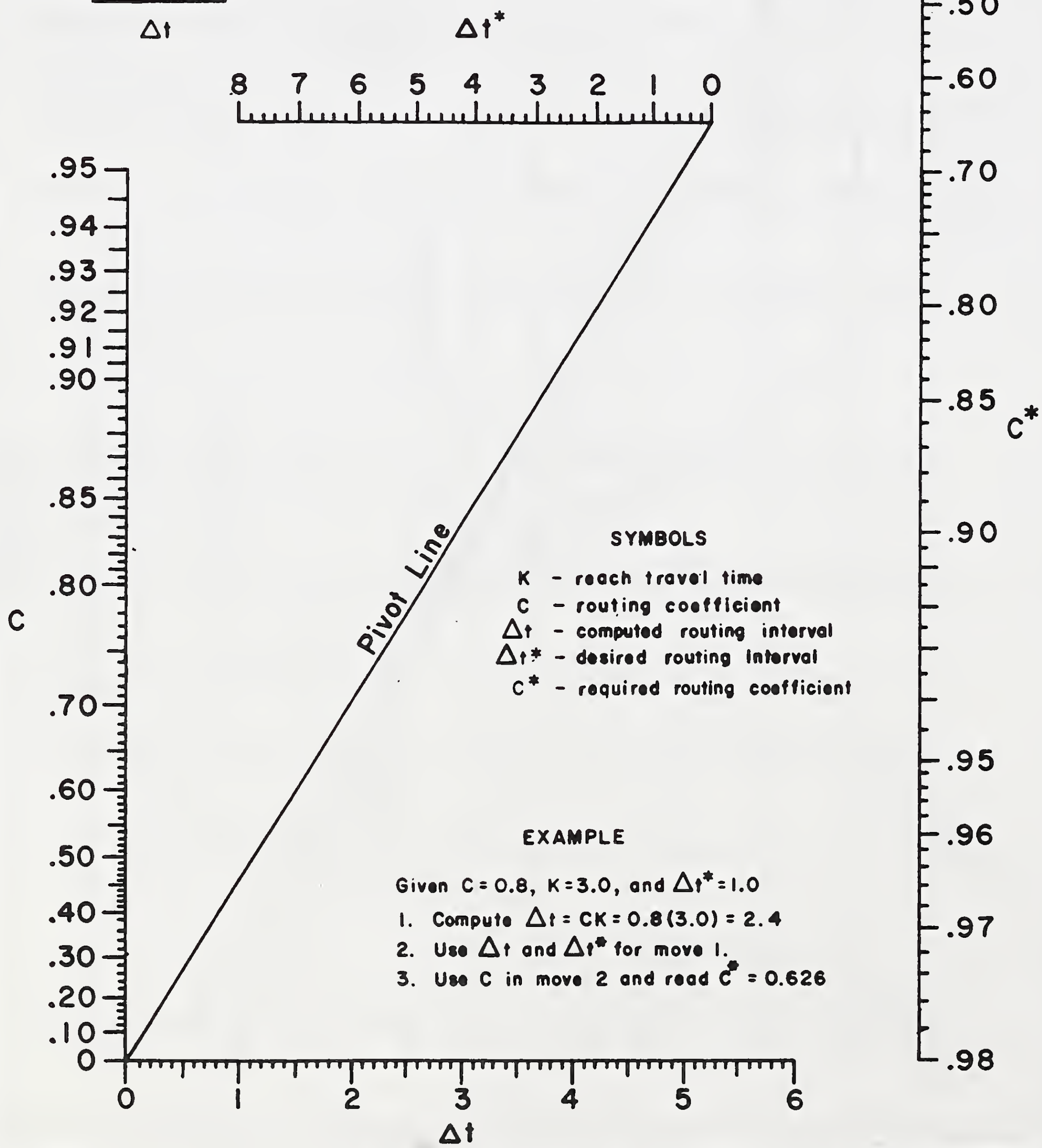
Figure 17-13. Convex routing coefficient versus velocity.

HYDROLOGY: NOMOGRAPH FOR SOLUTION OF $C^{*}=1-(1-C)^{\frac{\Delta t^{*}+\Delta t}{1.5\,\Delta t}}$



If $\Delta t^{*} > 8.0$ or $\Delta t > 6.0$ then divide both by 2 to find C^{*}. Example : $\Delta t^{*} = 3.0$ and $\Delta t = 8.2$. Use $\Delta t^{*} = 1.5$ and $\Delta t = 4.1$.

The Δt and Δt^{*} must be in identical units of time (minutes or hours or days, etc.)



SYMBOLS

K - reach travel time
C - routing coefficient
Δt - computed routing interval
Δt^{*} - desired routing interval
C^{*} - required routing coefficient

EXAMPLE

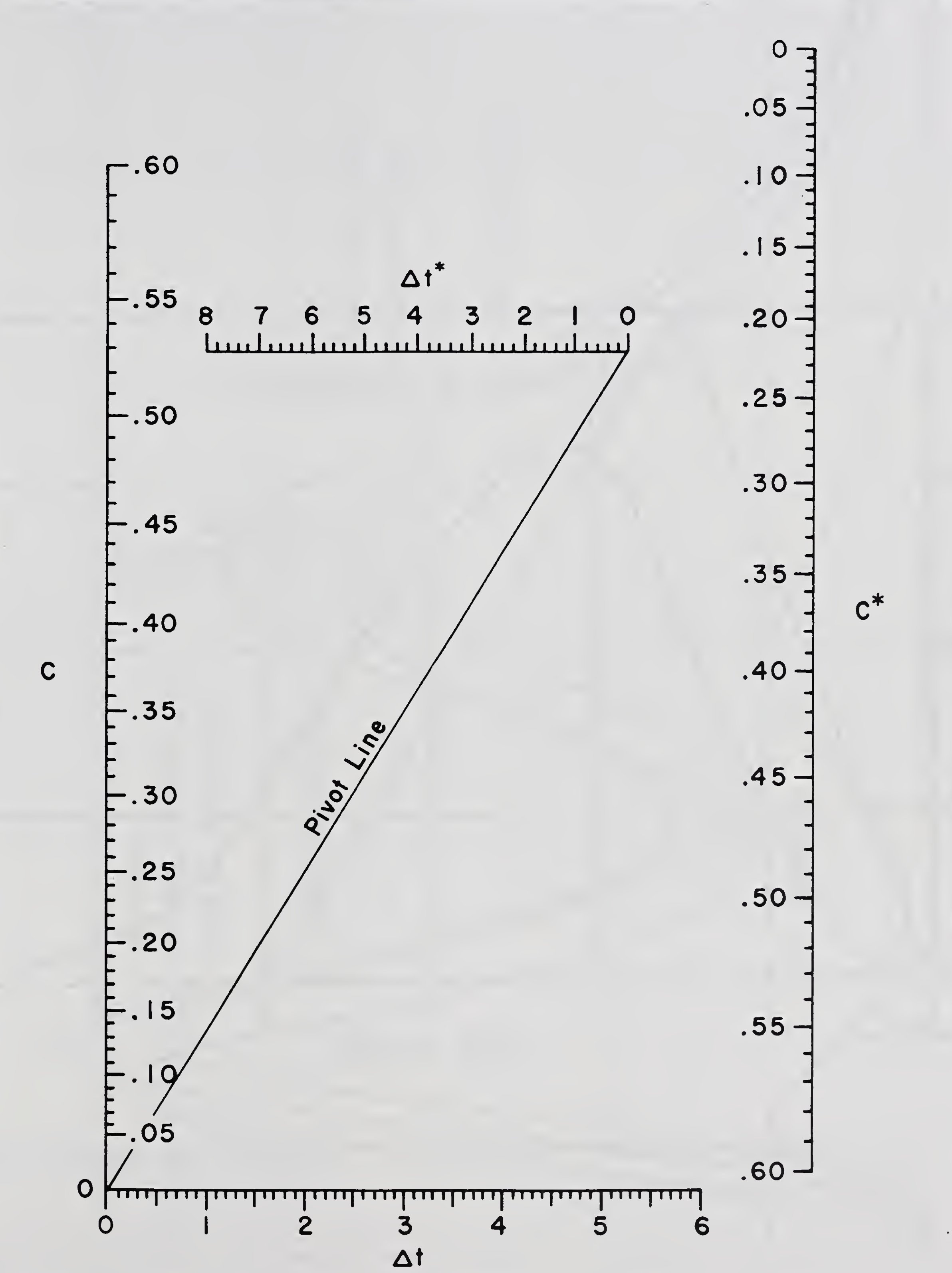
Given $C = 0.8$, $K = 3.0$, and $\Delta t^{*} = 1.0$

1. Compute $\Delta t = CK = 0.8(3.0) = 2.4$
2. Use Δt and Δt^{*} for move 1.
3. Use C in move 2 and read $C^{*} = 0.626$

REFERENCE	U. S. DEPARTMENT OF AGRICULTURE	STANDARD DWG. NO.
Equation by W. A. Styner	SOIL CONSERVATION SERVICE	ES- 1025 REV
Nomograph by Glenn G. Commons	ENGINEERING DIVISION - HYDROLOGY BRANCH	SHEET 1 OF 2
		DATE 6 - 71

Figure 17-14. ES-1025 rev. sheet 1 of 2.

HYDROLOGY: NOMOGRAPH FOR SOLUTION OF $C^{*}=1-(1-C)^{\frac{\Delta t^{*}+\Delta t}{1.5\,\Delta t}}$



REFERENCE	U. S. DEPARTMENT OF AGRICULTURE	STANDARD DWG. NO.
Equation by W. A. Styner	SOIL CONSERVATION SERVICE	ES-1025 REV
Nomograph by Glenn G. Commons	ENGINEERING DIVISION - HYDROLOGY BRANCH	SHEET 2 OF 2
		DATE 6 - 71

Figure 17-14. ES-1025 rev. sheet 2 of 2.

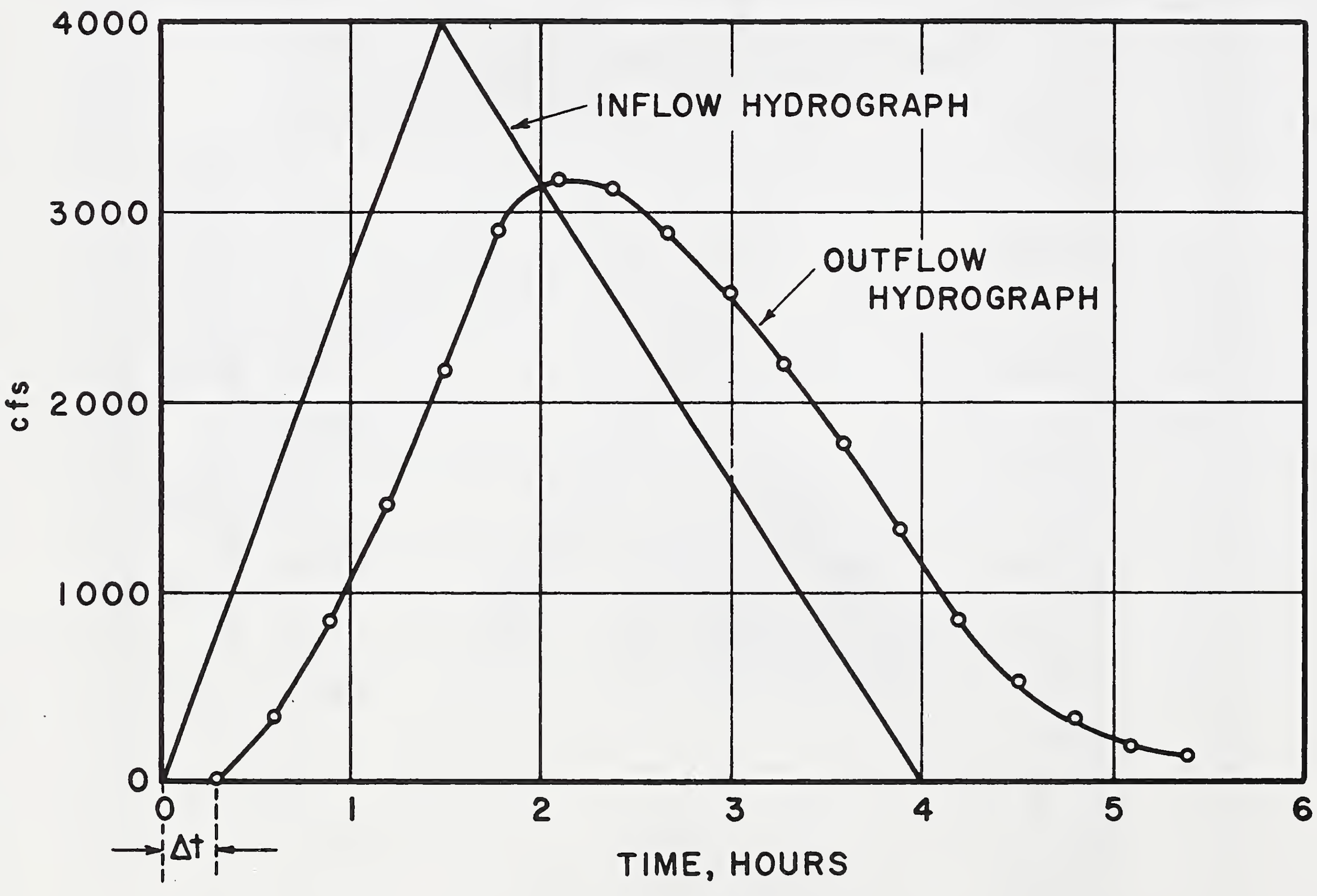


Figure 17-15. Inflow and routed outflow hydrograph for Example 17-7.

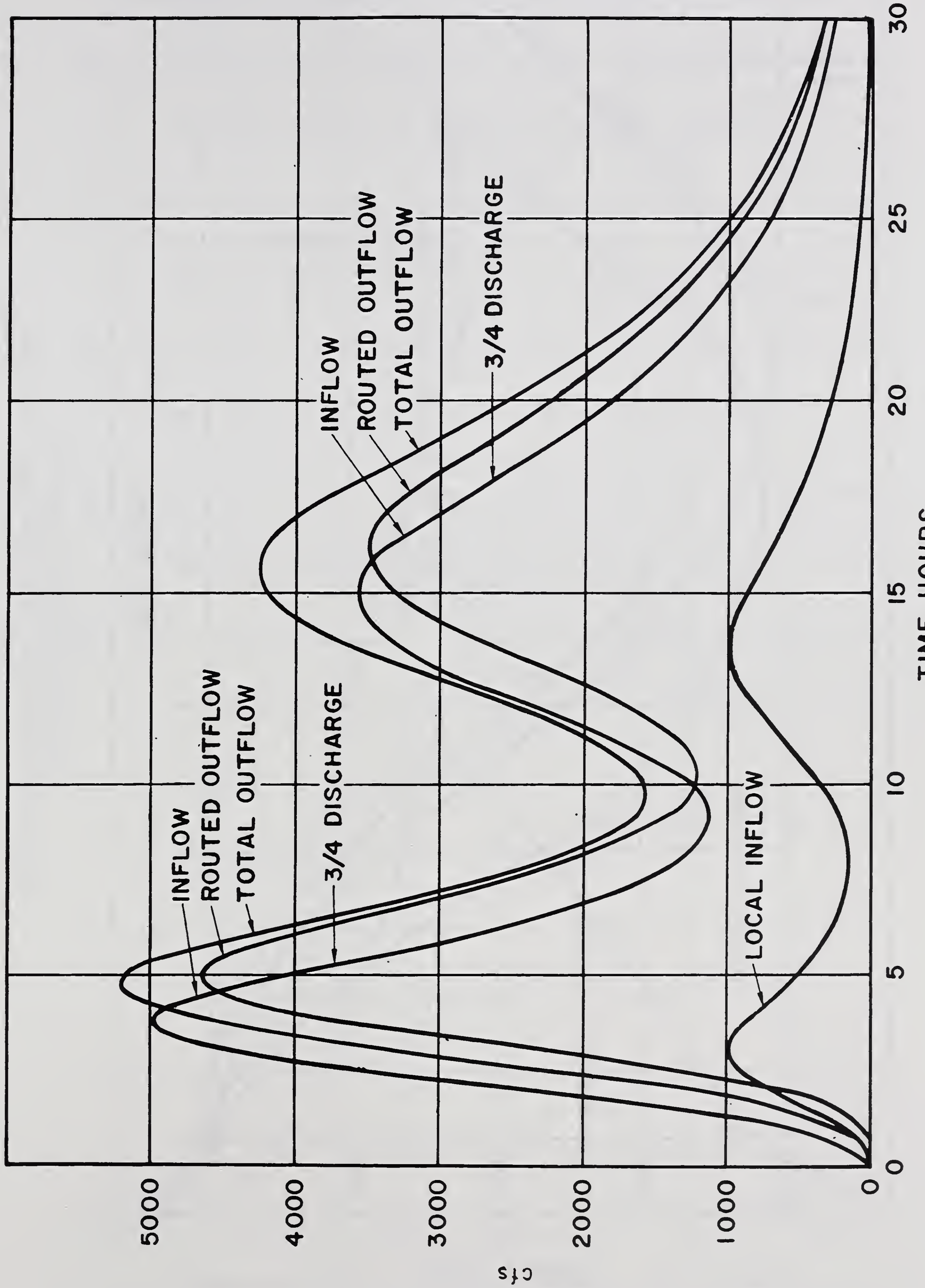


Figure 17-16. Inflow and routed outflow hydrograph for Example 17-8.

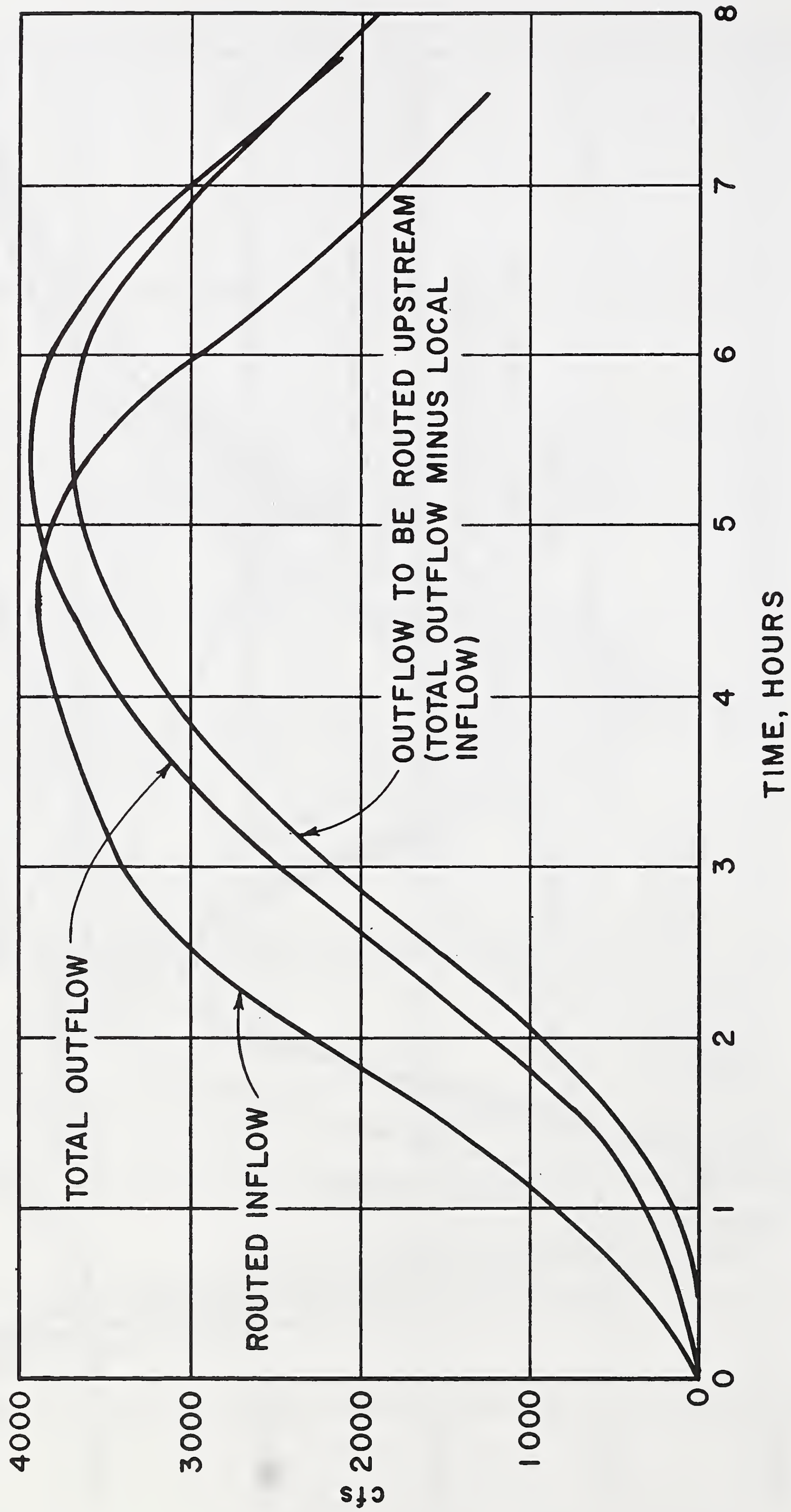


Figure 17-17. Outflow and routed inflow hydrograph for Example 17-9.

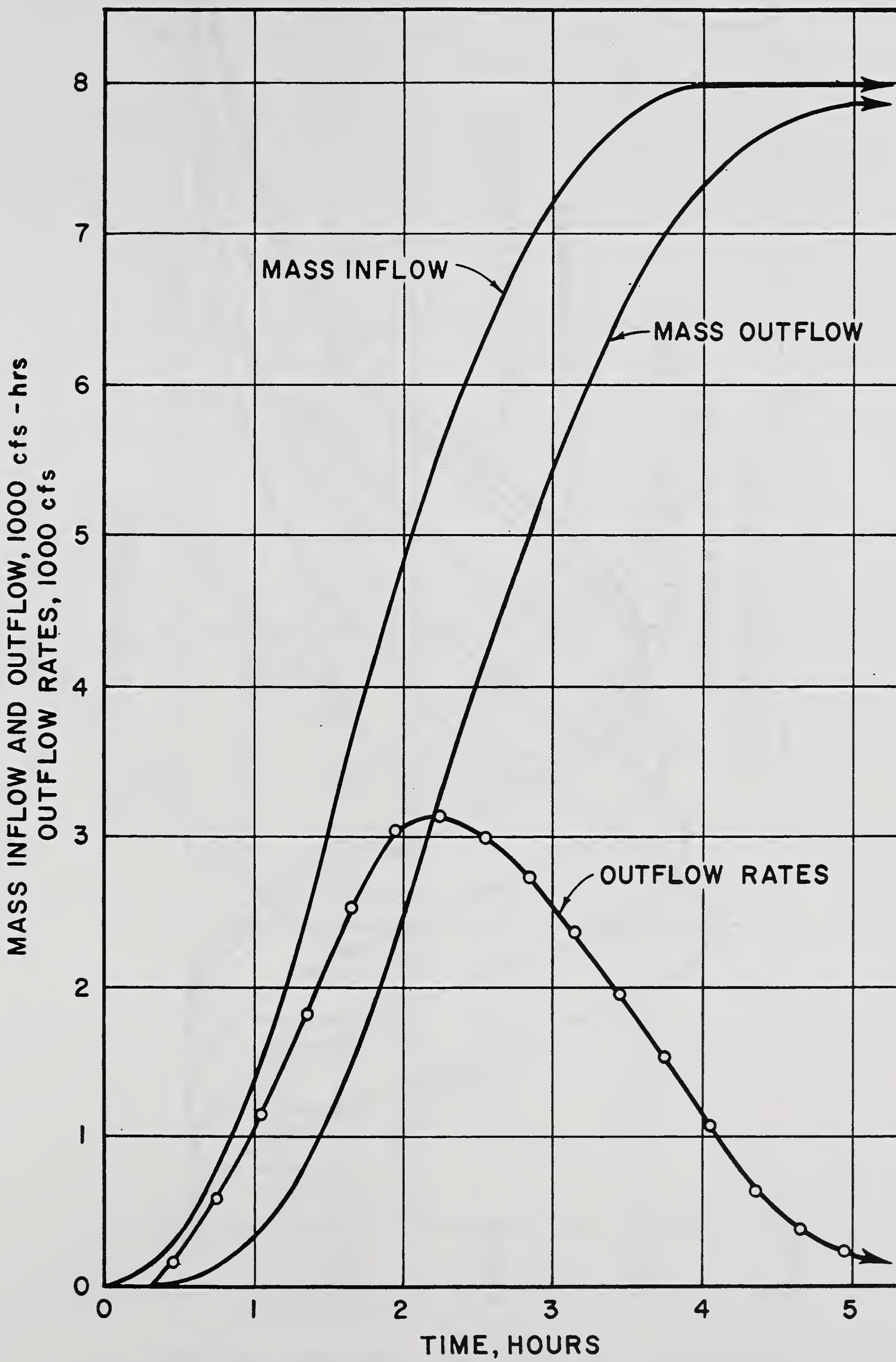


Figure 17-18. Mass inflow, mass outflow and rate hydrograph for Example 17-10.

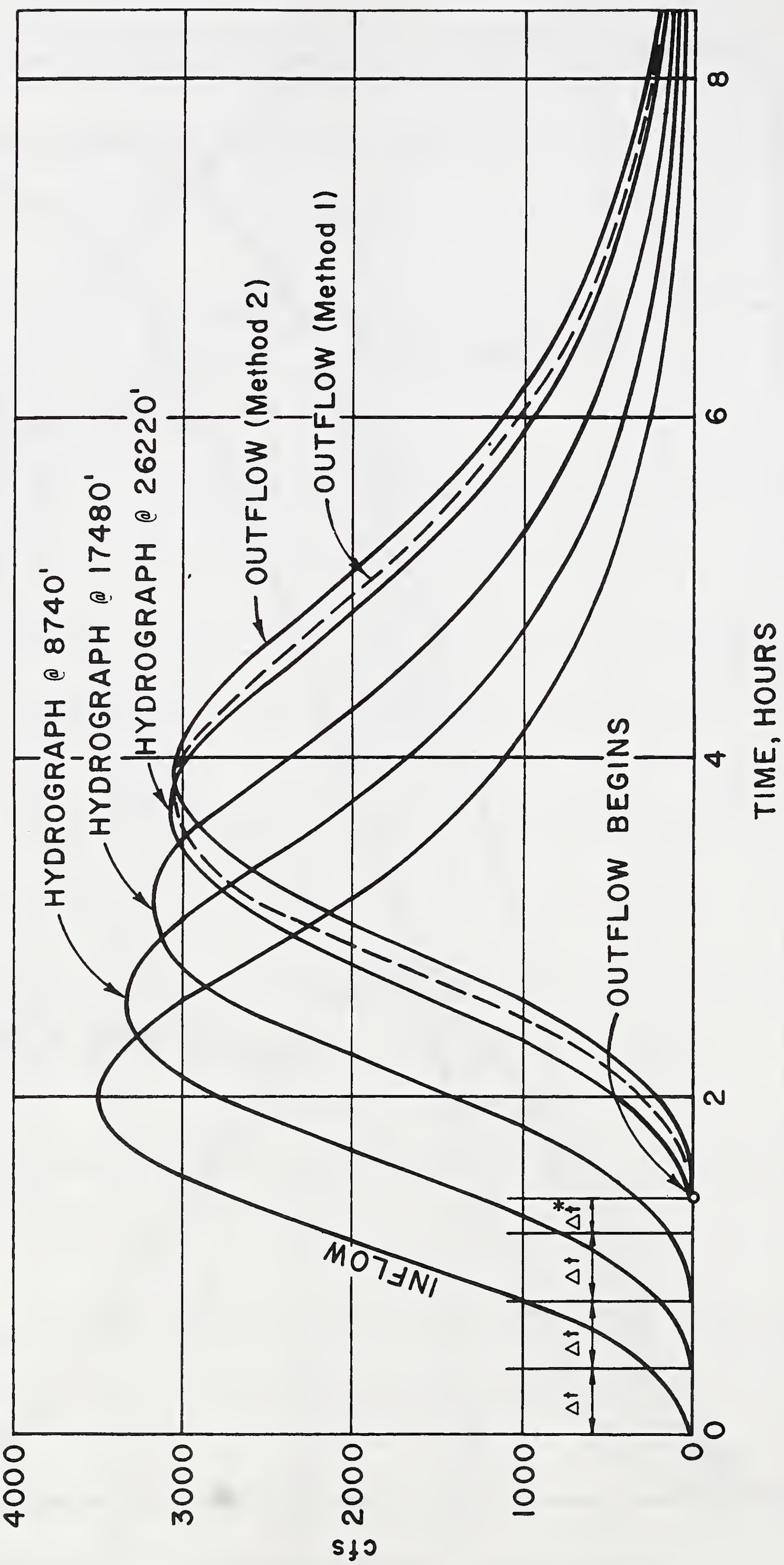


Figure 17-19. Inflow hydrograph and routed outflow hydrographs for Example 17-11, Method 1 and 2.

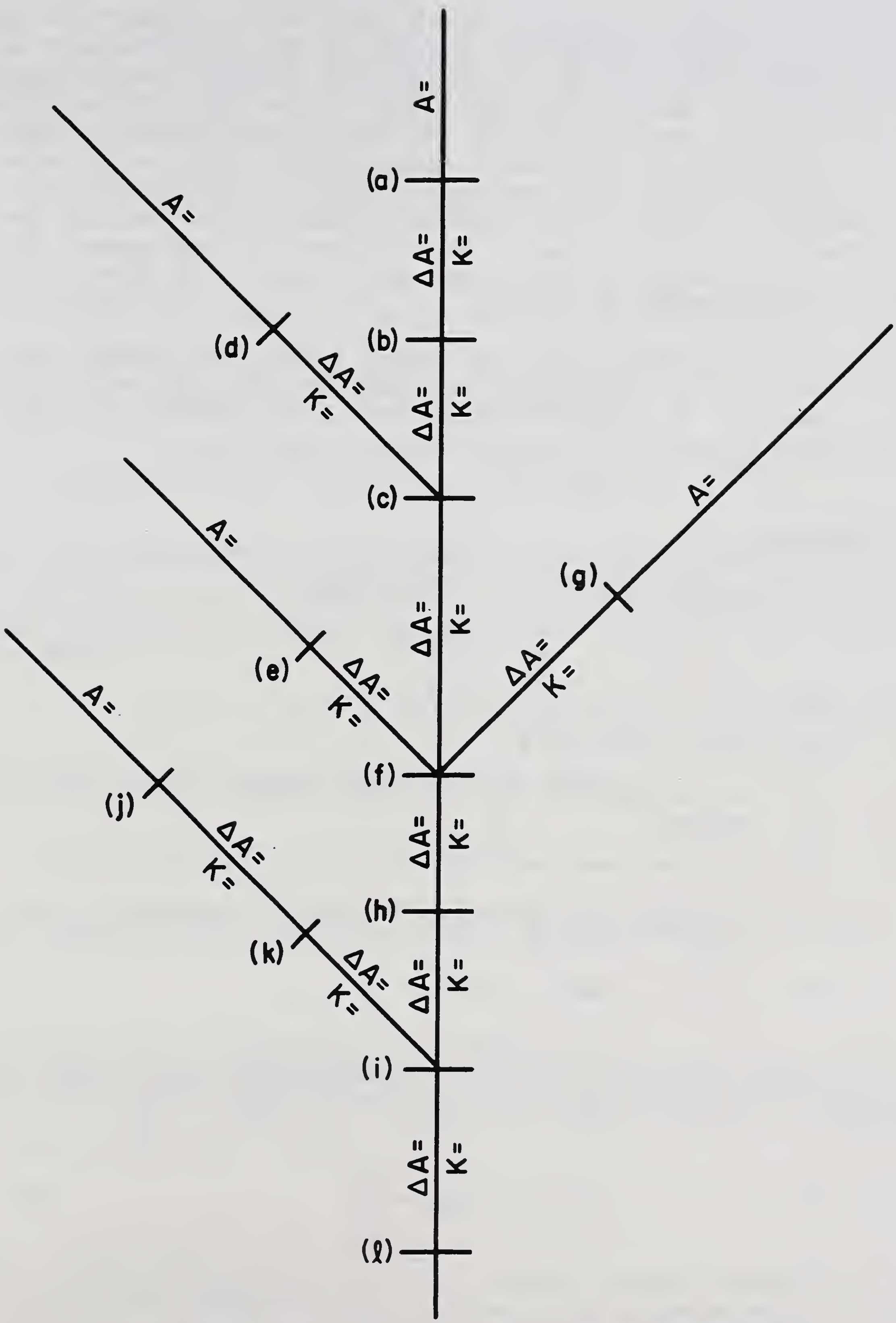


Figure 17-20. Typical Schematic diagram for routing through a system of channels.

Unit-Hydrograph Routing Methods

Principles of the unit hydrograph theory are given in Chapter 16. They apply to single-peaked hydrographs originating from uniform runoff on the contributing area but they can be extended to apply to more complex runoff conditions. Despite the limitations of the theory it has features that can be used in determining peak rates in stream reaches not only when the watershed is in a "present" unreservoired condition but also when it is controlled by many reservoirs. It is the ease with which complex systems of control structures are evaluated that has made the unit-hydrograph type of routing a popular method for many years. If suitable data are used the results are usually as good as those obtained by more detailed methods of routing.

In this part of the chapter the basic equations for unit-hydrograph routing will be given and discussed and some of their uses explained by means of examples. The unit-hydrograph method of routing gives only the peak rates of runoff. The peak-producing hydrograph, if it is needed, must be obtained in some other way.

Basic Equations

All of the unit-hydrograph working equations are derived from the relationship for the peak rate of a unit hydrograph:

$$q_p = \frac{K\ A\ Q}{T_p} \qquad \text{(Eq. 17-37)}$$

where
- q_p = peak rate in cfs
- K = a constant (not the routing parameter used in the Convex method)
- A = drainage area contributing runoff; in square miles
- Q = average depth of runoff, in inches, from the contributing area
- T_p = time to peak, in hours

By letting q_p, K, A, Q, and T_p stand for a watershed in one condition and using primed symbols q_p', K', A', Q', and T_p' for the same watershed in a condition being studied, then by use of Equation 17-37 it is evident that:

$$q_p = q_p \frac{A'\ Q'\ T_p}{A\ \ Q\ \ T_p'} \qquad \text{(Eq. 17-38)}$$

which is a typical working equation of the unit hydrograph method. It can be used, for example, in determining the peak rates after establishment of land use and treatment measures on a watershed. In such work the present peaks, areas, runoff amounts, and peak times are known and it is only a matter of finding the change in runoff by use of Chapter 10 methods. The areas and peak times are assumed to remain constant.

When a floodwater retarding structure, or other structure controlling a part of the watershed, is being used in the "future" condition then the value of A' is reduced. And if there are releases from the structure then they must also be taken into account. For a project having structures controlling a total of A* square miles and having an average release rate of q* csm, the peak rate equation becomes:

$$q'_p = q_p \frac{(A' - A^*)}{A} \frac{Q'}{Q} \frac{T_p}{T'_p} + q^* (A^*) \qquad \text{(Eq. 17-39)}$$

When using Equation 17-39 to find the reduced peak rate the major assumption is that the structures are about uniformly distributed over the watershed. Another assumption is that all structures contribute to q*, but this is sometimes too conservative an assumption (see the section titled "Use of Equation 17-43 on large watersheds").

When $A' = A$ and $T'_p = T_p$, which is the usual case when evaluating land use and treatment effects, Equation 17-38 becomes:

$$q'_p = q_p \frac{Q'}{Q} \qquad \text{(Eq. 17-40)}$$

which is one of the basic expressions of the unit hydrograph theory. If the same simplification applies when evaluating structures then Equation 17-39 becomes:

$$q'_p = q_p \frac{A - A^*}{A} + q^* (A^*) \qquad \text{(Eq. 17-41)}$$

Equation 17-41 can be further simplified by using:

$$r = \frac{A^*}{A} \qquad \text{(Eq. 17-42)}$$

where r is the fraction of drainage area under control or the percent of control divided by 100. Using Equation 17-42 in Equation 17-41 gives:

$$q'_p = q_p (1 - r) + q^*(A^*) \qquad \text{(Eq. 17-43)}$$

Effects of storm duration and time of concentration

When the effects of a change in either the storm duration or the time of concentration must be taken into account, one way to do it is to use the following relation from Chapter 16:

$$T_p = a(D) + b(T_c) \qquad \text{(Eq. 17-44)}$$

where T_p = time to peak, in hours

a = a constant

D = storm duration, in hours, during which runoff is generated; it is usually less than the total storm duration.

b = a constant

T_c = time of concentration, in hours

As shown in Chapter 16, the constants a and b can be taken as 0.5 and 0.6 respectively, for most problems, in which case Equation 17-44 becomes:

$$T_p = 0.5\ D + 0.6\ T_c \qquad \text{(Eq. 17-45)}$$

Using Equation 17-45 in equations 17-37, 17-38, and 17-39 produces working equations in which either the storm duration or the time of concentration can be changed and the effect of the change determined. Such equations are not often used because the main comparison is usually between present and future conditions in which only runoff amount and drainage area will change. In special problems where storm duration must be taken into account there are other approaches that are more applicable (see the section titled "Use of Equation 17-43 on large watersheds").

Elimination of T_p

In many physiographic areas there is a consistent relation between T_p and A because there is a typical storm condition or pattern. The relationship is usually expressed as:

$$T_p = c\ A^d \qquad \text{(Eq. 17-46)}$$

where c is a constant multiplier and d is a constant exponent. Substituting cA^d for T_p in Equation 17-37 gives:

$$q_p = k\ A^{(1 - d)}\ Q \qquad \text{(Eq. 17-47)}$$

where $k = K/c$. Letting $(1 - d) = h$, Equation 17-47 becomes:

$$q_p = k\ A^h\ Q \qquad \text{(Eq. 17-48)}$$

which is the working equation in practice. The parameters k and h are obtained from data for a large storm over the watershed or region being studied. Values of q_p at several locations are obtained either from streamflow stations or by means of slope-area measurements (Chapter 14): values of Q associated with each q_p are obtained from the station data or by use of rainfall and watershed data and methods of chapter 10; and drainage areas at each location are determined. A plotting of q_p/Q against A is made on log paper and a line of best fit is drawn through the plotting. The multiplier k is the intercept of the line where A = 1 square mile and the exponent h is the slope of the line. See the section titled "Use of Equations 17-48, 17-50, and 17-52" for an application of this procedure.

After h is known, the equivalent of Equation 17-38 is:

$$q_p' = q_p \left(\frac{A'}{A}\right)^h \frac{Q'}{Q} \qquad \text{(Eq. 17-49)}$$

The k's cancel out in making this change.

In the "Concordant Flow" method of peak determination, Equation 17-48 is modified to take into account the effects of control structures and their release rates, with the working equation being:

$$q_p' = k\ A^h\ Q\ (1 - r) + q^*(A^*) \qquad \text{(Eq. 17-50)}$$

or:

$$q_p' = q_p\ (1 - r) + q^*(A^*) \qquad \text{(Eq. 17-51)}$$

which is the same as Equation 17-43 in form but where q_p is now determined from Equation 17-48.

Equations 17-39, 17-41, 17-43, 17-50, and 17-51 should be used only when the storm runoff volume does not exceed the storage capacity of the structure with the smallest capacity. If the runoff does exceed that capacity these equations must be modified further. Equation 17-50, for example, becomes:

$$q_p' = k\ A^h\ (Q - r\ Qs) + q^*(A^*) \qquad \text{(Eq. 17-52)}$$

where Qs is the average storage capacity of the structures. It is shown in Example 17-20 how Equation 17-51 and similar equations can be used even when the capacity varies from structure to structure.

Working equations for special cases

Additional equations can be developed from those given if a special problem arises in watershed evaluation. For an example, suppose that Equation 17-43 is to be used for determining the effects of a proposed system of floodwater retarding structures in a watershed, and that the evaluation reaches are so long that the percent of area reservoired varies significantly from the head to the foot of the reach. To modify Equation 17-43 for this case, let A* be the area reservoired, A the total area, and r = A*/A for the head of the reach; and let B* be the total area reservoired (including A*), B the total area (including A), and r" = B*/B for the foot of the reach. For evaluations to be made at the foot of the reach, Equation 17-43 then becomes:

$$q_p' = q_p \left(\frac{2 - r - r''}{2}\right) + q^* \left(\frac{A^* + B^*}{2}\right) \qquad \text{(Eq. 17-53)}$$

After first computing (2 - r - r")/2 = C' and (A* + B*)/2 = C" for the reach, the working equation becomes:

$$q_p' = q_p\ C' + q^*\ C'' \qquad \text{(Eq. 17-54)}$$

where C' and C" are the computed coefficients. Each evaluation reach requires its own set of coefficients.

Examples
The problems in the following examples range from the very simple to the complex, the latter being given to show that unit-hydrograph methods have wide application. For some complex problems, however, it will generally be more efficient to use the SCS electronic-computer evaluation program.

Use of Equation 17-40. - This basic expression of the unit hydrograph theory has many uses. The major limitation in its use is that Q and Q' must be about uniformly distributed over the watershed being studied. The following is a typical but simple problem.

Example 17-12.--A watershed has a peak discharge of 46,300 cfs from a storm that produced 2.54 inches of runoff. What would the peak rate have been for a runoff of 1.68 inches?

1. Apply Equation 17-40.
For this problem q_p = 46,300 cfs, Q = 2.54 inches, and Q' = 1.68 inches. By Equation 17-40 q_p' = 46300(1.68/2.54) = 30,604 cfs, which is rounded to 30,600 cfs.

Use of Equation 17-43 - The major limitations in the use of this equation are that both the runoffs and the structures must be about uniformly distributed over the watershed and that the stream travel times for the "future" condition must be about the same as for the "present." The following is a typical but simple problem.

Example 17-13.--A watershed of 183 square miles has a flood peak of 37,800 cfs. If 42 square miles of this watershed were controlled by floodwater retarding structures having an average release rate of 15 csm, what would the reduced peak be?

1. Compute r.
By Equation 17-42 r = 42/183 = 0.230 because A* = 42 and A = 183 square miles.

2. Apply Equation 17-43.
For this problem, q_p = 37,800 cfs, r = 0.230 from step 1, q* = 15 csm, and A* = 42 square miles. By Equation 17-43 q_p' = 37800(1 - 0.230) + 15(42) = 29,736 cfs, which is rounded to 29,700 cfs. This is the reduced peak.

Use of Equation 17-43 on large watersheds.- If Equation 17-43 is used for evaluating the effects of structures in a large watershed or river basin the releases from structures far upstream may not add to the peak rates in the lower reaches of the main stem. And if releases from certain upstream structures do not affect peaks far downstream then those structures also are not reducing the peak rates, therefore their drainage areas should not be used in the equation.

In problems of this kind the approach to be taken is relatively simple though there are supplementary computations to be made before the equation is used. The key step in the approach is finding the T_p for an evaluation flood and using only those areas and structures close enough to the sub-basin outlet to affect the peak rate of that flood. How this is done will be illustrated using the data and computations of Table 17-21. The data are for a sub-basin of 620 square miles, with a time of concentration of 48 hours. Storm durations for the floods to be evaluated will vary from 1 to over 72 hours, which means that the sub-basin T_p will also vary considerably.

Table 17-21 is developed as follows:

Column 1 lists the travel times on the sub-basin main stem from the outlet point to selected points upstream, which are mainly junctions with major tributaries. The first entry is for the outlet point.

Column 2 gives the total drainage area above each selected point.

Column 3 gives the increments of area.

Column 4 gives the accumulated areas, going upstream. These are the contributing areas when the flood's T_p is within the limits shown in column 1. For example, when T_p is between 3.5 and 9.1 hours, the contributing drainage area is 74 square miles. T_p must be at least 48 hours before the entire watershed contributes to the peak rate.

Column 5 shows the total areas controlled by structures.

Column 6 gives the increments of controlled area.

Column 7 gives the accumulated controlled areas, going upstream.

Column 8 gives values of r, which are computed using entries of columns 7 and 4.

Column 9 gives values of (1 - r), which are computed using entries of column 8.

Column 10 gives the total average release rate in cfs for the controlled areas of column 7. For this table the average release rate q* is 7 csm. Therefore the q*(A*) entry for a particular row is the column 7 area of that row multiplied by the average rate in csm.

Only columns 1, 9, and 10 are used in the remaining work. To determine the effect of the structures the q_p and T_p of the evaluation flood must be known, the proper entries taken from the table, and Equation 17-43 applied. For example, if q_p = 87,000 cfs and T_p = 24 hours for a particular flood, first enter column 1 with T_p = 24 hours and find the row to be used, in this case it is between T_t values of 21.1 and 28.0 hours; next select (1 - r) = 0.459 from column 9 of that row and q*(A*) = 1,491 cfs from column 10; finally, use Equation 17-43 which gives q_p' = 87000 (0.459) + 1491 = 41,424 cfs, which is rounded to 41,400 cfs.

Table 17-21 Data and working table for use of Equation 17-43 on a large watershed

T_t (hrs)	A (sq.mi.)	ΔA (sq.mi.)	Au (sq.mi.)	A* (sq.mi.)	ΔA^* (sq.mi.)	A^*_u (sq.mi.)	r	(1 - r)	$q^*(A^*)$ 1/ (cfs)
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)
0	620			359					
		8	8		0	0	0	1.000	0
2.0	612			359					
		6	14		3	3	.214	.786	21
3.5	606			356					
		60	74		24	27	.365	.635	189
9.1	546			332					
		90	164		43	70	.426	.574	490
15.3	456			289					
		80	244		56	126	.517	.483	882
21.1	376			233					
		150	394		87	213	.541	.459	1491
28.0	226			146					
		101	495		77	290	.586	.414	2030
31.0	125			69					
		98	593		48	338	.570	.430	2366
42.0	27			21					
		27	620		21	359	.580	.420	2513
48.0	0			0					

1/ Using an average rate of q* = 7 csm.

If any other point in the sub-basin is also to be used for evaluation of structure effects then a separate table is needed for that point.

Use of Equations 17-48, 17-50, and 17-52.- When streamflow data or slope-area measurements and Q estimates are available for a watershed and its vicinity, the information can be used to construct a graph of q_p/Q and A as shown in Figure 17-21. This is the graphical form of Equation 17-48. If a line with an intercept of 484 cfs/in. and slope of 0.4 can be reasonably well fitted to the data, as in this case, it means that the hydrograph shapes of these watersheds closely resemble the shape of the unit hydrograph of Figure 16-1 (see Chapter 16). Usually the slope will be 0.4 for other shapes of hydrographs (the reason for this is discussed in Chapter 15) but the intercept will vary. For the line of Figure 17-21, Equation 17-48 can be written:

$$q_p = 484\ A^{0.4}\ Q \qquad \text{(Eq. 17-55)}$$

The following examples show some typical uses of the graph or its equation.

Example 17-14.--For a watershed in the region to which Figure 17-21 applies, A = 234 square miles and Q = 3.15 inches for a storm event. What is q_p?

1. Find q_p/Q for the given A.

Enter the graph with A = 234 square miles and at the line of relation find q_p/Q = 4,290 cfs/in.

2. Compute q_p.

Multiplying q_p/Q by Q gives q_p, therefore, q_p = 3.15(4290) = 13,500 cfs by a slide-rule computation.

If part of a watershed is controlled by floodwater retarding structures the graph can be used together with equation 17-50, as follows:

Example 17-15.--A watershed of 234 square miles has a system of floodwater retarding structures on it controlling a total of 103 square miles. Each structure has a storage capacity of 4.5 inches before discharge begins through the emergency spillway. Each structure has an average release rate of 15 csm. When the storm runoff Q is 4.1 inches what is the peak rate with (a) structures not in place, and (b) structure in place?

1. Determine the flood peak for the watershed with structures not in place.

Use the method of Example 17-14. Enter Figure 17-21 with A = 234 square miles and find q_p/Q = 4,290 cfs/in. Multiplying that result by Q = 4.1 inches gives q_p = 4.1(4290) = 17,600 cfs by a slide-rule computation. This discharge is ($k\ A^n\ Q$) in Equation 17-50.

2. Determine (1 - r).
From Equation 17-42 $r = A^*/A = 103/234 = 0.440$. Then $(1 - r) = 1 - 0.440 = 0.560$.

3. Determine the flood peak for the watershed with structures in place.
Use Equation 17-50 with the results of steps 1 and 2 and the given data for controlled area and release rate: $q_p' = 17600(0.560) + 15(103) = 11{,}410$ cfs, using a slide-rule for the multiplications. Round the discharge to 11,400 cfs.

If the storm runoff exceeds the storage capacities of the structures but the capacities are the same for all structures then Equation 17-52 can be applied as shown in the following example.

Example 17-16.--For the same watershed and structures used in Example 17-18 find the peak rates without and with structures in place when the storm runoff is 6.21 inches.

1. Determine the flood peak for the watershed with structures not in place.
Use the method of Example 17-14. Enter Figure 17-21 with A = 234 square miles and find $q_p/Q = 4{,}290$ cfs/in. This is $(k\ A^h)$ in Equation 17-52. Multiplying that result by Q = 6.21 inches gives $q_p = 6.21\ (4290) = 26{,}700$ cfs by a slide-rule computation. This is the peak rate without structures in place.

2. Determine r.
From Equation 17-42 $r = A^*/A = 103/234 = 0.440$

3. Determine the flood peak for the watershed with structures in place.
Use Equation 17-52 with $(k\ A^h) = 4{,}290$ cfs/in. from step 1; Q = 6.21 inches, as given; r = 0.440, from step 2; and Qs = 4.5 inches, $q^* = 15$ csm, and $A^* = 103$ square miles as given in Example 17-17. Then $q_p' = 4290(6.21 - 0.440(4.5)) + 15(103) = 18160 + 1540 = 19{,}700$ cfs.

Note that the effect of the release rate on reducing the storm runoff amount is not taken into account in this example. This means that the peak of 19,700 cfs is slightly too large and that this approach gives a conservatively high answer.

If the storage capacities of the structures vary then Equation 17-52 is used with (Q - r Qs) computed by a more detailed method, as shown in the following example.

Example 17-17.--A watershed of 311 square miles has a system of floodwater retarding structures controlling a total of 187 square miles and having average release rates of 8 csm. Storage capacities of the structures are shown in column 3 of Table 17-22; these are the capacities before emergency spillway discharge begins. When the storm runoff is

uniformly 7.5 inches over the watershed, what is the peak rate of flow with (a) no structures in place and (b) structures in place?

1. Determine the flood peak for the watershed with structures not in place.
Use the method of Example 17-14. Enter Figure 17-21 with A = 311 square miles and find q_p/Q = 4,800 cfs/in. This is ($k\ A^h$) in Equation 17-52. Multiplying that result by Q = 7.5 inches gives q_p = 36,000 cfs by a slide-rule computation. This is the peak rate without structures in place.

2. Compute the equivalent of (r Qs) in Equation 17-52.
The factor (r Qs) can also be expressed as:

$$(r\ Qs) = \frac{\Sigma(A_x \times Qs_x)}{A} \qquad \text{(Eq. 17-56)}$$

where A_x is the drainage area in square miles of the x-th structure and Qs_x is the reservoir capacity in inches for that structure. In Table 17-22 each drainage area of column 2 is multiplied by the respective storage of column 3 to get the entry for column 4. But note that when the storage exceeds the storm runoff it is the storm runoff amount, in this case 7.5 inches, which is used to get the entry for column 4. Equation 17-56 is solved for (r Qs) by dividing the sum of column 4 by the total watershed area:

$$(r\ Qs) = \frac{967.26}{311} = 3.11 \text{ inches}$$

(Note: Column 4 is not needed if the calculations are made by accumulative multiplication on a desk-calculator.)

3. Determine the flood peak for the watershed with structures in place.
Use Equation 17-52 with ($k\ A^h$) = 4,800 cfs/in. from step 1; Q = 7.5 inches, as given; (r Qs) = 3.11 inches as computed in step 2; and q^* = 8 csm and A^* = 187 square miles, as given. This gives: q_p' = 4800(7.5 - 3.11) + 8(187) = 21100 + 1495 = 22,595 cfs, which is rounded to 22,600 cfs. This is the peak rate with structures in place.

DISCUSSION. These examples are a sample of the many ways in which the unit-hydrograph method of routing can be used. Accuracy of the method depends on what has been ignored, such as variable release rate, surcharge storage, and so on. In general, the method gives conservative results--that is, the effects of structures, for example, are usually underestimated so that the peak rate is slightly too high.

The examples also show that as the problem contains more details the procedure gets more complex. It is easily possible to make this "short-cut" method so complicated it becomes difficult to get the solution. For this reason, and for reasons of accuracy, it is better to use the SCS electronic-computer program for complex routing problems.

Table 17-22 Area and storage data for Example 17-17.

Floodwater retarding structure	Contributing drainage area (sq. mi.)	Storage (in.)	A_x x Qs_x (sq. mi. x in.)
(1)	(2)	(3)	(4)
1	14.2	6.1	86.62
2	8.3	6.8	56.44
3	3.7	9.2	21.75*
4	9.4	5.5	51.70
5	17.1	4.5	76.95
6	25.2	3.7	93.24
7	12.9	5.1	65.79
8	6.0	7.5	45.00
9	3.2	10.0	24.00*
10	5.5	8.0	41.25*
11	21.0	4.0	84.00
12	16.4	4.3	70.52
13	9.3	6.5	60.45
14	11.6	5.5	63.80
15	12.5	5.3	66.25
16	10.7	5.0	53.50
		$\Sigma(A_x \text{ x } Qs_x)$ =	967.26

* This is (drainage area) x (storm runoff of 7.5 inches) because the storage greater than the runoff is ineffective and should not be used in the computation.

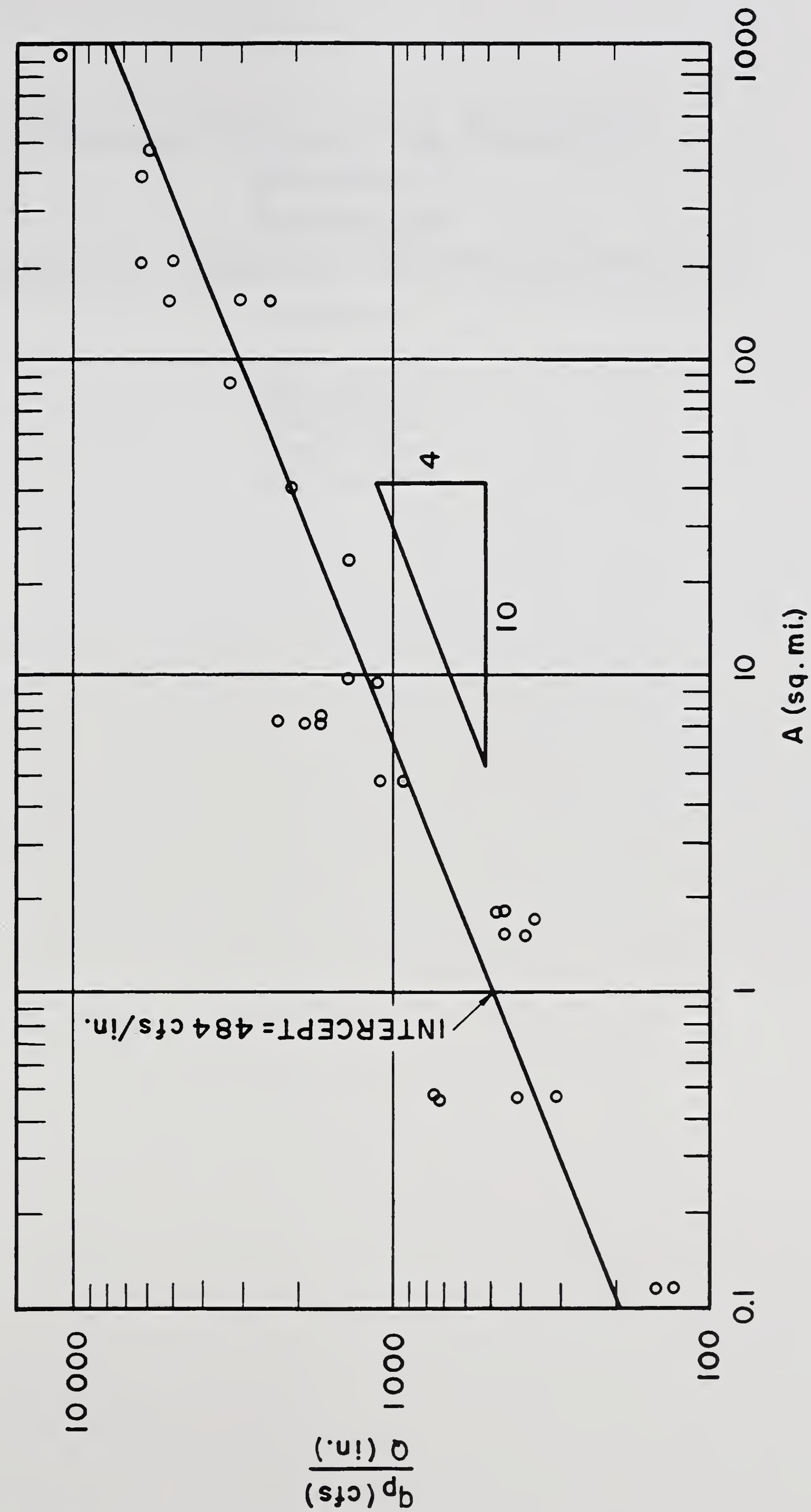


Figure 17-21. q_p/Q versus A for a typical physiographic area.

National Engineering Handbook
Section 4
Hydrology
Chapter 18. Selected Statistical Methods

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Chapter 18
Selected Statistical Methods

Introduction

Chapter 18 is a guide for applying selected statistical methods to solve hydrologic problems. The chapter includes a review of basic statistical concepts, a discussion of selected statistical procedures, and references to procedures in other available documents. Examples illustrate how statistical procedures apply to typical problems in hydrology.

In project evaluation and design, the hydrologist or engineer must estimate the frequency of individual hydrologic events. This is necessary when making economic evaluations of flood protection projects; determining floodways; and designing irrigation systems, reservoirs, and channels. Frequency studies are based on past records and, where records are insufficient, on simulated data.

Meaningful relationships sometimes exist between hydrologic and other types of data. The ability to generalize about these relationships may allow data to be transferred from one location to another. Some procedures used to perform such transfers, called regionalization, are covered in this chapter.

The examples in this chapter contain many computer-generated tables. Some table values (especially logarithmic transformations) may not be as accurate as values calculated by other methods. Numerical accuracy is a function of the number of significant digits and the algorithms used in data processing, so some slight differences in numbers may be found if examples are checked by other means.

Basic Data Requirements

Basic Concepts

To analyze hydrologic data statistically, the user must know basic definitions and understand the intent and limitations of statistical analysis. Because collection of all data (entire population) from a physical system is usually not feasible and recorded data from the system may be limited, observations must be based on a *sample* that is representative of the population.

Statistical methods are based on the assumption of randomness, which implies an event cannot be predicted with certainty. By definition, *probability* is an indicator of the likelihood of the occurrence of an event and is measured on a scale from 0 to 1, with 0 indicating no chance of occurrence and 1 indicating certainty of occurrence.

Events or values that do not occur with certainty are often called random variables. There are two types of random variables, discrete and continuous. A *discrete random variable* is one that can only take on values that are whole numbers. For example, the outcome of a toss of a die is a discrete random variable because it can only take on the integer values 1 to 6. The concept of risk as it is applied in frequency analysis is also based on a discrete probability distribution. A *continuous random variable* can take on values defined over a continuum; for example, peak discharge takes on values other than discrete integers.

A function that defines the probability that a random value will occur is called a *probability distribution function*. For example, the log-Pearson Type III distribution, often used in frequency analyses, is a probability distribution function. A probability *mass function* is used for discrete random variables while a *density function* is used for continuous random variables. If values of a distribution function are added (discrete) or integrated (continuous), then a *cumulative distribution function* is formed. Usually, hydrologic data that are analyzed by frequency analysis are presented as a cumulative distribution function.

Types of Data

The application of statistical methods in hydrologic studies requires measurement of physical phenomena. The user should understand how the data are collected and processed before they are published. This knowledge helps the user assess the accuracy of the data. Some types of data used in hydrologic studies include rainfall, snowmelt, stage, streamflow, temperature, evaporation, and watershed characteristics.

Rainfall is usually measured as an accumulated depth over a period of time. Measurements represent the amount caught by the gage opening and are valid only for the gage location. The amount collected may be affected by gage location and physical factors near the gage. Application over large areas requires a study of adjacent gages and determinations of a weighted rainfall amount. More complete discussions of rainfall collection and evaluation procedures are found in Chapter 4 of this handbook section.

Snowfall is measured as depth or as water equivalent on the ground. As with rainfall, the measurement represents only the depth at the measurement point. The specific gravity of the snow times the depth of the snow determines the water equivalent of the snowpack, which is the depth of water that would result from melting the snow. To use snow information for such things as predicting water yield, the user should thoroughly know snowfall, its physical characteristics, and its measurement. National Engineering Handbook, Section 22, "Snow Survey and Water Supply Forecasting" (1972) further discusses these subjects.

Stages are measurements of the elevation of the water surface as related to an established datum, either the channel bottom or mean sea level, called National Geodetic Vertical Datum (NGVD). Peak stages are measured by nonrecording gages, crest-stage gages, or recording gages. Peak stages from nonrecording gages may be missed because continuous visual observations are not available. Crest-stage gages record only the maximum gage height and recording gages provide a continuous chart or record of stage.

Streamflow or discharge rates are extensions of the stage measurements that have been converted through the use of rating curves. Discharge rates indicate the runoff from the drainage area above the gaging station and are expressed in cubic feet per second (cfs). Volume of flow past a gage, expressed as a mean daily or hourly flow (cfs-days or cfs-hours), can be calculated if the record is continuous. Accuracy of streamflow data depends largely on physical features at the gaging site, frequency of observation, and the type and adequacy of the equipment used. Flows can be affected by upstream diversion and storage. U.S. Geological Survey Water Supply Paper 888 (Corbett 1962) further discusses streamflow data collection.

Daily temperature data are usually available, with readings published as maximum, minimum, and mean measurements for the day. Temperatures are recorded in degrees Fahrenheit or degrees Celsius. National Weather Service, Observing Handbook No. 2, Substation Observations (1972), describes techniques used to collect meteorological data.

Evaporation data are usually published as pan evaporation in inches per month. Pan evaporation is often adjusted to estimate gross lake evaporation. The National Weather Service has published pan evaporation values in "Evaporation Atlas for the Contiguous 48 United States" (Farnsworth, Thompson, and Peck 1982).

Watershed characteristics used in hydrologic studies include drainage area, channel slope, geology, type and condition of vegetation, and other features. Maps, field surveys, and studies are used to obtain this information. Often data on these physical factors are not published, but the U.S. Geological Survey maintains a file on watershed characteristics for most streamgage sites. Many federal and state agencies collect and publish hydrometeorological data (table 18-1). Many other organizations collect hydrologic data that are not published but may be available upon request.

Table 18-1.—Sources of basic hydrologic data collected by federal agencies

Agency	Data					
	Rainfall	Snow	Stream-flow	Evapo-ration	Air temp.	Water stage
Agricultural Research Service	X	X	X	X	X	X
Corps of Engineers	X	X	X	X		X
Forest Service	X	X	X		X	X
U.S. Geological Survey (WATSTORE)		X	X	X	X	
International Boundary & Water Commission	X		X	X	X	X
River Basin Commissions	X		X			X
Bureau of Reclamation	X	X	X	X	X	X
Soil Conservation Service	X	X	X		X	X
Tennessee Valley Authority	X		X	X	X	X
National Climatic Data Center, NOAA	X	X		X	X	X

Data Errors

The possibility of instrumental and human error is inherent in data collection and publication for hydrologic studies. Instrumental errors are caused by the type of equipment used, its location, and conditions at the time measurements are taken. Instrumental errors can be accidental if they are not constant or do not create a trend, but they may also be systematic if they occur regularly and introduce a bias into the record. Human errors by the observer or by others who process or publish the information can also be accidental or systematic. Examples of human errors include improper operation or observation of equipment, misinterpretation of data, and errors in transcribing and publishing.

The user of the hydrologic data should be aware of the possibility of errors in observations and should recognize observations that are outside the expected range of values. Knowledge of the procedures used in collecting the data is helpful in recognizing and resolving any questionable observations, but the user should consult the collection agency when data seem to be in error.

Types of Series

Hydrologic data are usually presented in chronological order. If all the data for a certain increment of observation (for example, daily readings) are presented for the entire period of record, this is a *complete-duration series.* Many of these data do not have significance and can be excluded from hydrologic studies. The complete-duration series is only used for duration curves or mass curves. From the complete-duration series two types of series are selected, the partial-duration series and the extreme-event series.

The *partial-duration series* includes all events in the complete-duration series with a magnitude above a selected base for high events or below a selected base for low events. Unfortunately, independence of events that occur in a short period is hard to establish because long-lasting watershed effects from one event can influence the magnitude of succeeding events. Also, in many areas the extreme events occur during a relatively short period during the year. Partial-duration frequency curves are developd either by graphically fitting the plotted sample data or by using empirical coefficients to convert the partial-duration series to another series.

The *extreme-event series* includes the largest (or smallest) values from the complete-duration series, with each value selected from an equal time interval in the period of record. If the time interval is taken as 1 year, then the series is an *annual series,* for example, a tabulation of the largest peak flows in each year through the period of record as an annual peak flow series at the location. Several high peak flows may occur within the same year, but the annual peak series includes only the largest peak flow per year. Table 18-2 illustrates both a partial-duration and annual peak flow series.

Table 18-2.—Flood peaks for East Fork Big Creek near Bethany, Mo. (06897000)[1]

Year	Peaks above base (cfs)	Year	Peaks above base (cfs)	Year	Peaks above base (cfs)	Year	Peaks above base (cfs)
1940	1,780*	1947	2,240	1958	1,780*	1967	1,640
	1,120		8,120*		1,780		3,350*
			2,970				1,640
1941	2,770		3,700	1959	3,800		
	2,950*		4,920		3,000	1968	3,150*
					1,500		
1942	1,190	1948	1,260		2,660	1969	2,990
	1,400		2,310*		5,100*		3,110*
	925				3,660		1,730
	925	1949	2,000*		2,280		2,910
	1,330				1,890		2,270
	1,330	1950	1,160				2,060
	5,320		1,300*	1960	2,280		
	6,600*				4,650	1970	2,090
		1951	1,090		1,960		3,070*
1943	958		2,920*		1,680		2,060
	1,680		1,090		4,740*		
	2,000		1,720		2,040	1971	2,000*
	3,110*		2,030				
	925		1,060	1961	1,760	1972	3,190*
	2,470		1,000		1,520		
	1,330				3,100		
	1,190	1952	1,440		5,700*		
	2,240		1,610		2,300		
	3,070		1,090				
			1,230	1962	2,630		
1944	1,120		2,970*		2,750		
	3,210*		2,280		1,760		
	2,620				1,820		
	2,170	1953	925*		3,880*		
1945	3,490	1954	1,330*	1963	2,100*		
	4,120*						
	2,310	1955	1,500	1964	1,880		
	2,350		2,240*		1,910*		
			1,500				
1946	4,400			1965	1,730		
	1,520	1956	1,560		3,480*		
	1,720		2,500*				
	6,770*			1966	2,430*		
	1,960	1957	1,620*				

[1] Partial-duration base is 925 cfs, the lowest annual flood for this series.
Annual series values are starred (*). Data from USGS Water Supply Papers.

Some data indicate seasonal variation, monthly variation, or causative variation. Major storms or floods may occur consistently during the same season of the year or may be caused by more than one factor, for example, by rainfall and snowmelt. Such data may require the development of a series based on a separation by causative factors or a particular time frame.

Data Transformation

In many instances, complex data relationships require that variables be transformed to approximate linear relationships or other relationships with known shapes. Types of data transformation include:

1. Linear transformation, which involves addition, subtraction, multiplication, or division by a constant.
2. Inverted transformation by use of the reciprocal of the data variables.
3. Logarithmic transformation by use of the logarithms of the data variables.
4. Exponential transformation, which includes raising the data variables to a power.
5. Any combination of the above.

The appropriate transformation may be based on a physical system or may be entirely empirical. All data transformations have limitations. For example, the reciprocal of data greater than +1 yields values between 0 and +1. And logarithms, which are commonly used in hydrologic data, can only be derived from positive data.

Distribution Parameters and Moments

A probability distribution function, as previously defined, is represented by a mathematical formula that includes one or more of the following parameters: *location*, which provides reference values for the random variable; *scale*, which characterizes the relative dispersion of the distribution; and *shape*, which describes the outline or form of a distribution.

A parameter is *unbiased* if the average of estimates taken from repeated samples of the same size converges to the population value. A parameter is *biased* if the average estimate does not converge to the population value.

A probability density function can be characterized by its moments, which are also used in characterizing data samples. In hydrology, three moments of special interest are mean, variance, and skew.

The first moment about the origin is the *mean*, a location parameter that measures the central tendency of the data and is computed by:

$$\overline{X} = \frac{1}{N}\left(\sum_{i=1}^{N} X_i\right) \tag{18-1}$$

where $\overline{X}$ is the sample arithmetic mean having N observations and X_i is the i^{th} observation of the sample data.

The remaining two moments of interest are taken about the mean instead of the origin. The first moment about the mean is always zero.

The *variance*, a scale parameter and the second moment about the mean, measures the dispersion of the sample elements about the mean. The unbiased estimate of the variance (S^2) is given by:

$$S^2 = \left[\frac{1}{N-1}\sum_{i=1}^{N}(X_i - \overline{X})^2\right] \tag{18-2}$$

A biased estimate of the variance results when the divisor $(N - 1)$ is replaced by N. An alternative form for computing the unbiased sample variance is given by:

$$S^2 = \frac{1}{N-1}\left[\sum_{i=1}^{N} X_i^2 - \frac{1}{N}\left(\sum_{i=1}^{N} X_i\right)^2\right] \tag{18-3}$$

This equation is often used for computer application because it does not require prior computation of the mean. But, because of the sensitivity of equation 3 to the number of significant digits carried through the computation, equation 2 is often preferred.

The *standard deviation* (S) is the square root of the variance and is used more frequently than the variance because its units are the same as those of the mean.

The *skew*, a shape parameter and the third moment about the mean, measures the symmetry of a distribution. The sample skew (G) can be computed by:

$$G = \frac{N}{(N-1)(N-2)S^3}\left[\sum_{i=1}^{N}(X_i - \overline{X})^3\right] \tag{18-4}$$

Although the range of the skew is theoretically unlimited, there is a mathematical limit, based on sample size, that limits the possible skew (Kirby 1974). A skew of zero indicates a symmetrical distribution. Another equation for computing skew that does not require prior computation of the mean, is:

$$G = \frac{N^2\left(\sum_{i=1}^{N} X_i^3\right) - 3N\left(\sum_{i=1}^{N} X_i\right)\left(\sum_{i=1}^{N} X_i^2\right) + 2\left(\sum_{i=1}^{N} X_i\right)^3}{N(N-1)(N-2)S^3} \quad (18\text{-}5)$$

This equation is extremely sensitive to the number of significant digits used during computation and may not give an accurate estimate of the sample skew.

Basic Concepts

Frequency analysis is a statistical method commonly used to analyze a single random variable. Even when the population distribution is known, uncertainty is associated with the occurrence of the random variable. When the population is unknown, there are two sources of uncertainty: randomness of future events and accuracy of estimation of the relative frequency of occurrence. The cumulative density function is estimated by fitting a frequency distribution to the sample data. A *frequency distribution* is a generalized cumulative density function of known shape and range of values.

The probability scale of the frequency distribution differs from the probability scale of the cumulative density function by the relation (1 − p) where:

$$p + q = 1 \quad (18\text{-}6)$$

The variables p and q represent the accumulation of the density function for all values *less than* and *greater than*, respectively, the value of the random variable. The accumulation is made from the right end of the probability density function curve when one considers high values such as peak discharge. U.S. Department of Agriculture, Soil Conservation Service, Technical Release 38 (1976) presents the accumulation of the Pearson III density function for both p and q for a range of skew values.

When minimum values (p) such as low flows are considered, the accumulation of the probability density function is from the left end of the curve. The resulting curve represents values *less than* the random variable.

Plotting Positions and Probability Paper

Statistical computations of frequency curves are independent of how the sample data are plotted, so the data should be plotted along with the calculated frequency curve to verify that the general trend of the data reasonably agrees with the frequency distribution curve.

Various plotting formulas are used and many are of the general form:

$$PP = \frac{100(M - a)}{N - a - b + 1} \quad (18\text{-}7)$$

where PP is the plotting position for a value in percent chance; M is the ordered data (largest to smallest for

maximum values and smallest to largest for minimum values); N is the size of the data sample; and a and b are constants. Constants of some commonly used plotting position formulas are:

Name	*a*	*b*
Weibull	0	0
Hazen	$-M+1$	$-N+M$
California	0	1
Blom	$3/8$	$3/8$

The Weibull plotting position is used to plot the sample data in the chapter examples:

$$PP = \frac{100(M)}{N+1} \tag{18-8}$$

Each probability distribution has its own probability paper for plotting. The probability scale is defined by transferring a linear scale of standard deviates (K values) into probabilities for that distribution. The frequency curve for a distribution will be a straight line on paper specifically designed for that distribution.

Probability paper for logarithmic normal and extreme value distributions is readily available. Distributions with a varying shape statistic (i.e., log-Pearson III and gamma) require paper with a different probability scale for each value of the shape statistic. For these distributions a special plotting paper is not practical. The log-Pearson III and gamma distributions are usually plotted on logarithmic normal probability paper. The plotted frequency line may be curved, but this is more desirable than developing a new probability scale each time these distributions are plotted.

Probability Distribution Functions

Normal

The normal distribution, used to evaluate continuous random variables, is symmetrical and bell-shaped. The range of the random variable is $-\infty$ to $+\infty$. Two parameters (location and scale) are required to fit the distribution. These parameters are approximated by the sample mean and standard deviation. The normal distribution is the basis for much of statistical theory, but generally does not fit hydrologic data.

The log-normal distribution (normal distribution with logarithmically transformed data) is often used in hydrology to fit high or low discharge data or in regionalization analysis. Its range is 0 to $+\infty$. Example 18-1 illustrates the development of a log-normal distribution curve.

Pearson III

Karl Pearson developed a system of 12 distributions that can approximate all forms of single-peak statistical distributions. The system includes three main distributions and nine transition distributions, all of which were developed from a single differential equation. The distributions are continuous but can be fitted to various forms of discrete data sets (Chisman 1968).

The type III (negative exponential) is the distribution frequently used in hydrologic analysis. It is nonsymmetrical and is used with continuous random variables. The probability density function can take on many shapes. Depending on the shape parameter, the random variable range can be limited on the lower end, the upper end, or both. Three parameters are required to fit the Pearson type III distribution. The location and scale parameters (mean and standard deviation) are the same as the normal distribution. The shape (or third) parameter is approximated by the sample skew.

When a logarithmic transformation is used, a lower bound of zero exists for all shape parameters. The log-Pearson type III is used to fit high and low discharge values, snow, and volume duration data. Example 18-1 illustrates the development of a log-Pearson type III distribution curve.

Two-Parameter Gamma

The two-parameter gamma distribution is nonsymmetrical and is used with continuous random variables to fit high- and low-volume duration, stage, and discharge data. Its probability density function has a lower limit of 0 and a defined upper limit of ∞. Two parameters are required to fit the distribution: β, a scale parameter, and γ, a shape parameter. A detailed description of how to fit the distribution with the two parameters and incomplete gamma function tables can be found in TP-148 (Sammons 1966). As a close approximation of this solution, a three-parameter Pearson type III fit can be made and TR-38 tables used. The mean and γ must be computed and converted to standard deviation and skewness parameters. Greenwood and Durand (1960) provide a method to calculate an approximation for γ that is a function of the relationship (R) between the arithmetic mean and geometric mean (G_m) of the sample data:

$$G_m = [X_1(X_2)(X_3)\ldots(X_N)]^{1/N} \tag{18-9}$$

$$R = \ln\left[\frac{\overline{X}}{G_m}\right] \qquad (18\text{-}10)$$

where ln is the natural logarithm.

a) If $0 \leq R \leq 0.5772$

$$\gamma = R^{-1}(0.5000876 + 0.1648852R - 0.0544274R^2) \qquad (18\text{-}11)$$

b) If $0.5772 \leq R \leq 17.0$

$$\gamma = \frac{8.898919 + 9.059950R + 0.9775373R^2}{R(17.79728 + 11.968477R + R^2)} \qquad (18\text{-}12)$$

c) If $R > 17.0$ the shape approaches a log-normal distribution, and a log-normal solution may be used.

The standard deviation and skewness can now be computed from γ and the mean:

$$S = \frac{\overline{X}}{\sqrt{\gamma}} \qquad (18\text{-}13)$$

$$G = \frac{2}{\sqrt{\gamma}} \qquad (18\text{-}14)$$

Extreme Value

The extreme value distribution, another nonsymmetrical distribution used with continuous random variables, has three main types. Type I is unbounded; type II is bounded on the lower end; and type III is bounded on the upper end. The type I (Fisher-Tippett) is used by the National Weather Service in precipitation analysis. Other federal, state, local, and private organizations also have publications based on extreme value theory.

Binomial

The binomial distribution, used with discrete random variables, is based on four assumptions:

1. The random variable may have only one of two responses (for example, yes or no, successful or unsuccesful, flood or no flood).
2. There will be n trials in the sample.
3. Each trial will be independent.
4. The probability of a response will be constant from one trial to the next.

The binomial distribution is used in assessing risk, which is discussed later in the chapter.

Cumulative Distribution Curve and TR-38

Selected percentage points on the cumulative distribution curve for normal, Pearson III, or gamma distributions can be computed with the sample mean, standard deviation, and skewness. TR-38 contains standard deviate (K_p) values for various values of skewness and probabilities. The equation used to compute points along the cumulative distribution curve is:

$$Q = \overline{X} + K_p S \qquad (18\text{-}15)$$

where Q is the random variable value at a selected exceedance probability, $\overline{X}$ is the sample mean, and S is the sample standard deviation. If a logarithmic transformation has been applied to the data, then the equation becomes:

$$\log Q = \overline{X} + K_p S \qquad (18\text{-}16)$$

where $\overline{X}$ and S are based on the moments of the logarithmically transformed sample data. With the mean, standard deviation, and skew computed, a combination of TR-38 and either equation 15 or 16 is used to calculate specified points along the cumulative distribution curve.

Data Considerations in Analysis

Outliers

If the population model is correct, outliers are population elements that occur but are highly unlikely to occur in a sample of a given size. Therefore, outliers can be due to sampling variation or to the use of the incorrect probability model. After the most likely probability model is selected, outlier tests can be performed for evaluating extreme events.

Outliers can be detected by use of test criteria in exhibit 18-1. Critical standard deviates (K_n values) for the normal distribution can be taken from the exhibit. Critical K values for other distributions are computed from the probability levels listed in the exhibit 18-1. Critical K values are used in either equation 15 or 16, along with sample mean and standard deviation, to determine an allowable range of sample element values.

The detection process is iterative: (1) use sample statistics, $\overline{X}$ and S, and K, with equation 15 or 16 to

detect a single outlier; (2) delete detected outliers from the sample; (3) recompute sample statistics without the outliers; and (4) begin again at step (1). Continue the process until no outliers are detected. High and low outliers can exist in a sample data set.

Two extreme values of about the same magnitude are not likely to be detected by this outlier detection procedure. In these cases, delete one value and check to see if the remaining value is an outlier. If the remaining value is an outlier, then both values should be called outliers or neither value should be called an outlier.

The detection process depends on the distribution of the data. A positive skewness indicates the possibility of high outliers, and a negative skewness indicates the possibility of low outliers. Thus, samples with a positive skew should be tested first for high outliers, and samples with negative skew should be tested first for low outliers.

If one or more outliers are detected, another frequency distribution should be considered. If a frequency distribution is found that appears to have fewer outliers, repeat the outlier detection process. If no better model is found, treat the outliers in the following order of preference:

1. Reduce their weight or impact on the frequency curve.
2. Eliminate the outliers from the sample.
3. Retain the outliers in the sample.

When historic data are available, high outlier weighting can be reduced by use of Appendix 6, Water Resources Council (WRC) Bulletin #17B (1981). If such data are not available, you must decide whether to retain or delete the high outliers. This decision involves judgment concerning the impact of the outliers on the frequency curve and its intended use. Low outliers can be given reduced weighting by treating them as missing data as outlined in Appendix 5, WRC Bulletin #17B.

Although WRC Bulletin #17B was developed for peak flow frequency analysis, many of the methods are applicable to other types of data.

Mixed Distributions

A mixed distribution occurs when at least two events in the population are due to different causes. In flow frequency analysis, a sample of annual peak discharges at a given site can be drawn from a single distribution or mixture of distributions. A mixture occurs when the series of peak discharges are caused by various types of runoff-producing events such as generalized rainfall, local thunderstorms, hurricanes, snowmelt, or any combination of these.

Previously discussed frequency analysis techniques may be valid for mixed distributions. If the mixture is due to a single or small group of values, these values may appear as outliers. After these values are identified as outliers, the sample can then be analyzed. However, if the number of values departing from the trend of the data becomes significant, a second trend may be evident. Two or more trends may be evident when the data are plotted on probability paper.

Populations with multiple trends will cause problems in analysis. The skewness of the entire sample will be greater than the skewness of samples that are separated by cause. The larger skewness will cause the computed frequency curve to differ from the sample data plot in the region common to both trends.

The two methods that can be used to develop a mixed distribution frequency curve are illustrated in example 18-3. The preferred method (method 1) involves separating the sample data by cause, analyzing the separated data, and combining the frequency curves. The detailed procedure is as follows:

1. Determine the cause for each annual event. If a specific cause cannot be found for each event, method 1 cannot be used.
2. Separate the data into individual series for each cause found in step 1. Some events may be common to more than one series and, therefore, belong to more than one series. For example, snowmelt and generalized rainfall could form an event that would belong to both series.
3. Collect the data that are necessary to form an annual series for each cause. Some series will not have an event for each year—for example, a hurricane series in an area where hurricanes occur about once every 10 years. If insufficient data for any series are a problem, then the method will need a truncated series with conditional probability adjustment. See Appendix 5, WRC Bulletin #17B.
4. Compute the statistics and frequency curve for each annual series separately.
5. Use the addition rule of probability to combine the computed frequency curves.

$$P\{A \cup B\} = P\{A\} + P\{B\} - [P\{A\} \times P\{B\}] \quad (18\text{-}17)$$

where $P\{A \cup B\}$ is the probability of an event of given magnitude occurring from either or both series, $P\{A\}$ and $P\{B\}$ are the probabilities of an event of given magnitude occurring from each series, and $[P\{A\} \times$

P{Bj] is the probability of an event from each series occurring in a single year.

An alternative method (method 2) that requires only the sample data may be useful in estimating the frequency curve for $q \leq 0.5$. This method is less reliable than method 1 and requires that at least the upper one-half of the data be generally normal or log normal if log-transformed data are used. A straight line is fitted to at least the upper half of the frequency range of the series. The standard deviation and mean are developed by use of the expected values of normal order statistics. The equations are:

$$S = \left[\frac{\left(\sum_{i=1}^{N} X_i^2\right) - \left(\sum_{i=1}^{N} X_i\right)^2 / n}{\left(\sum_{i=1}^{N} K_i\right) - \left(\sum_{i=1}^{N} K_i\right)^2 / n} \right]^{0.5} \qquad (18\text{-}18)$$

$$\overline{X} = \left(\sum_{i=1}^{N} X_i\right) - S\left(\sum_{i=1}^{N} K_i\right)^2 / n \qquad (18\text{-}19)$$

where n is the number of elements in the truncated series and K_i is the expected value of normal order statistics for the i^{th} element of the complete sample. Expected values of normal order statistics are shown in exhibit 18-2.

Incomplete Record and Zero Flow Years

An *incomplete record* refers to a sample in which some data are missing either because they were too low or too high to record or because the measuring device was out of operation. In most instances the agency collecting the data provides estimates for missing high flows. When the missing high values are estimated by someone other than the collecting agency, it should be documented, and the data collection agency advised. Most agencies do not routinely provide estimates of low flow values. The procedure that accounts for missing low values is a conditional probability adjustment explained in Appendix 5 of WRC Bulletin #17B.

Data sets containing zero values present a problem when one uses logarithmic transformations. The logarithm of zero is undefined and cannot be included. When a logarithmic transformation is desired, zeros should be treated as missing low data.

Historic Data

At many locations there is information about major hydrologic occurrences either before or after the period of systematic data collection. Such information, called *historic data*, can be used to adjust the frequency curve. The historic data define an extended time period during which rare events, either recorded or historic, have occurred. Historic data may be obtained from other agencies, from newspapers, or by interviews. A procedure for incorporating historic data into the frequency analysis can be found in Appendix 6 of WRC Bulletin #17B.

Frequency Analysis Reliability

The following discussion, which originally appeared in U.S. Corps of Engineers, Hydrologic Engineering Methods, Volume 3, Hydrologic Frequency Analysis (1975), concisely covers the main points of frequency reliability, including examples based on flood frequencies.

> The reliability of frequency estimates is influenced by:
> a) The amount of information available.
> b) The variability of the events.
> c) The accuracy with which the data were measured.
>
> In general with regard to item a, errors of estimate are inversely proportional to the square root of the number of independent items contained in the frequency array. Therefore, errors of estimates based on 40 years of record would normally be half as large as errors of estimates based on 10 years of record, other conditions being the same.
>
> The variability of events in a record (item b) is usually the most important factor affecting the reliability of frequency estimates. For example, the ratio of the largest to the smallest annual flood of record on the Mississippi River at Red River Landing, Louisiana, is about 2.7, whereas the ratio of the largest to the smallest annual flood of record on the Kings River at Piedra, California, is about 100, or 35 times as great. Statistical studies show that as a consequence of this factor, a flow corresponding to a given frequency that can be estimated within 10 percent on the Mississippi River, can be estimated only within 40 percent on the Kings River.
>
> The accuracy of data measurement (item c) normally has relatively small influence on the realiability of a frequency estimate, because such errors ordinarily are not systematic and tend to cancel, and because the influence of chance events is great in comparison with that of measurement errors. For this reason, it is usually better to include an estimated magnitude for a major flood; for example, that was not recorded because of gage failure, rather than to omit it from the frequency array, even though its magnitude can only be estimated approximately. However, it is advisable always to use the most reliable sources of data and, in particular, to guard against systematic errors such as result from using an unreliable rating curve.
>
> It should be remembered that the possible errors in estimating flood frequencies are very large, principally because

of the chance of having a nonrepresentative sample. Sometimes the ocurrence of one or two abnormal floods can change the apparent exceedance frequency of a given magnitude from once in 1,000 years to once in 200 years. Nevertheless, the frequency-curve technique is considerably better than any other tool available for some purposes and represents a substantial improvement over using an array restricted to observed flows only.

Effects of Watershed Modification

The analysis of streamflow data is complicated by the fact that watershed conditions are rarely constant during the period of record. Fire, floods, changing land use, channel modification, reservoir construction, and land treatment all contribute to changes in the hydrologic responses of a watershed. If the changes are significant, then standard statistical procedures cannot be used to develop the frequency curve.

Outline of Frequency Analysis Procedures

A. Obtain site information, historic data, and systematic data.
  1. Examine record period for changes in physical conditions. Use only data that are from periods of constant physical conditions (homogeneous).
  2. Estimate missing high data. The effort expended in estimating data depends on the use of the final frequency analysis.
  3. Obtain historic information.

B. Plot sample data.
  1. Use normal (logarithmic normal) probability paper.
  2. Observe general trend of plotted data.
  3a. For single-trend data, select the distribution that best defines the population from which the sample is drawn.
  3b. For multiple-trend data, use one of the mixed distribution techniques.

C. Compute frequency curve.
  1. Use sample statistics and distribution tables (such as TR-38).
  2. Plot curve on the paper with sample data.
  3. Compare general shape of curve with sample data. If the computed curve does not fit the data, check for outliers or for another distribution that may fit the population.

D. Detect outliers.
  1. Check for outliers according to the value of skewness, high first for positive skewness and low first for negative skewness.
  2. Delete outliers and recompute sample statistics.
  3. Continue the process until no outliers remain in sample.

E. Treat outliers and missing, low, and zero data.
  1. Check another frequency distribution model.
  2. For high outliers,
    a. if historical data are available, use Appendix 6 WRC Bulletin #17B.
    b. if historic data are not available, decide whether outliers should be retained in the sample.
  3. For low outliers and missing, low, and zero data, use Appendix 5, WRC Bulletin #17B.

F. Check reliability of results.
  1. Frequency curve estimates are based on prior experience and should be used with caution.
  2. Uncertainty of estimates increases as estimated values depart from the mean.

Example 18-1.—Development of log-normal and log-Pearson III frequency curves.

Annual peak discharge data for East Fork San Juan River near Pagosa Springs, Colo. (Station 09340000), are analyzed. Table 18-3 contains the water year (column 1) and annual peak values (column 2). Other columns in the table will be referenced by number in parentheses in the following steps:

1. Plot the data. Before plotting the data, arrange them in descending order (column 6). Compute Weibull plotting positions, based on a sample size of 44, from equation 8 (column 7), and then plot the data on logarithmic normal probability paper (fig. 18-1).

2. Examine the trend of plotted data. The plotted data follow a single trend that is nearly a straight line, so a log-normal distribution should provide an adequate fit. The log-Pearson type III distribution will also be included because it is computational, like the log normal.

3. Compute the required statistics. Use common logarithms to transform the data (column 3). Compute the sample mean by using the summation of sample data logarithms and equation 1:

$$\overline{X} = \frac{130.1245}{44} = 2.957376$$

Table 18-3.—Basic statistics data for example 18-1 (Station 09340000 E. Fork San Juan River near Pagosa Springs, Colo. Drainage area = 86.9 sq.mi. Elevation = 7,597.63 feet)

Water year (1)	Peak (cfs) (2)	X = Log(peak) (3)	$(\overline{X} - X)^2$ (4)	$(\overline{X} - X)^3$ (5)	Ordered peak (cfs) (6)	Weibull plot position 100 M/(N+1) (7)
1935	1480.0	3.170260	0.0453200	0.0096479	2460.0	2.2
1936	931.0	2.968948	0.0001339	0.0000015	2070.0	4.4
1937	1120.0	3.049216	0.0084347	0.0007747	1850.0	6.7
1938	1670.0	3.222715	0.0704052	0.0186813	1670.0	8.9
1939	580.0	2.763427	0.0376161	−0.0072956	1550.0	11.1
1940	606.0	2.782472	0.0305914	−0.0053505	1510.0	13.3
1941	2070.0	3.315969	0.1285889	0.0461111	1480.0	15.6
1942	1330.0	3.123850	0.0277137	0.0046136	1410.0	17.8
1943	830.0	2.919077	0.0014668	−0.0000562	1340.0	20.0
1944	1410.0	3.149218	0.0368034	0.0070604	1330.0	22.2
1945	1140.0	3.056904	0.0099059	0.0009859	1320.0	24.4
1946	590.0	2.770850	0.0347917	−0.0064895	1270.0	26.7
1947	724.0	2.859737	0.0095332	−0.0009308	1270.0	28.9
1948	1510.0	3.178975	0.0491064	0.0108819	1170.0	31.1
1949	1270.0	3.103803	0.0214409	0.0031395	1140.0	33.3
1950	463.0	2.665580	0.0851447	−0.0248449	1120.0	35.6
1951	709.0	2.850645	0.0113914	−0.0012158	1070.0	37.8
1952	1850.0	3.267170	0.0959725	0.0297318	1050.0	40.0
1953	1050.0	3.021188	0.0040720	0.0002598	1030.0	42.2
1954	550.0	2.740361	0.0470952	−0.0102203	934.0	44.4
1955	557.0	2.745853	0.0447416	−0.0094638	931.0	46.7
1956	1170.0	3.068185	0.0122787	0.0013606	923.0	48.9
1957	1550.0	3.190331	0.0542680	0.0126420	880.0	51.1
1958	1030.0	3.012836	0.0030758	0.0001706	865.0	53.3
1959	388.0	2.588830	0.1358257	−0.0500580	856.0	55.6
1960	865.0	2.937015	0.0004146	−0.0000084	856.0	57.8
1961	610.0	2.785329	0.0296001	−0.0050926	830.0	60.0
1962	880.0	2.944481	0.0001663	−0.0000021	820.0	62.2
1963	490.0	2.690195	0.0713854	−0.0190728	776.0	64.4
1964	820.0	2.913813	0.0018977	−0.0000827	724.0	66.7
1965	1270.0	3.103803	0.0214409	0.0031395	709.0	68.9
1966	856.0	2.932472	0.0006202	−0.0000154	610.0	71.1
1967	1070.0	3.029383	0.0051850	0.0003734	606.0	73.3
1968	934.0	2.970345	0.0001682	0.0000022	600.0	75.6
1969	856.0	2.932472	0.0006202	−0.0000154	590.0	77.8
1970	2460.0	3.390934	0.1879728	0.0814972	580.0	80.0
1971	515.0	2.711805	0.0603047	−0.0148090	557.0	82.2
1972	422.0	2.625311	0.1102667	−0.0366157	550.0	84.4
1973	1340.0	3.127104	0.0288077	0.0048895	515.0	86.7
1974	490.0	2.690195	0.0713854	−0.0190728	490.0	88.9
1975	1320.0	3.120572	0.0266331	0.0043464	490.0	91.1
1976	923.0	2.965200	0.0000612	0.0000005	463.0	93.3
1977	600.0	2.778150	0.0321219	−0.0057571	422.0	95.6
1978	776.0	2.889860	0.0045583	−0.0003078	388.0	97.8
	Summation	130.1245	1.659318	0.023534		

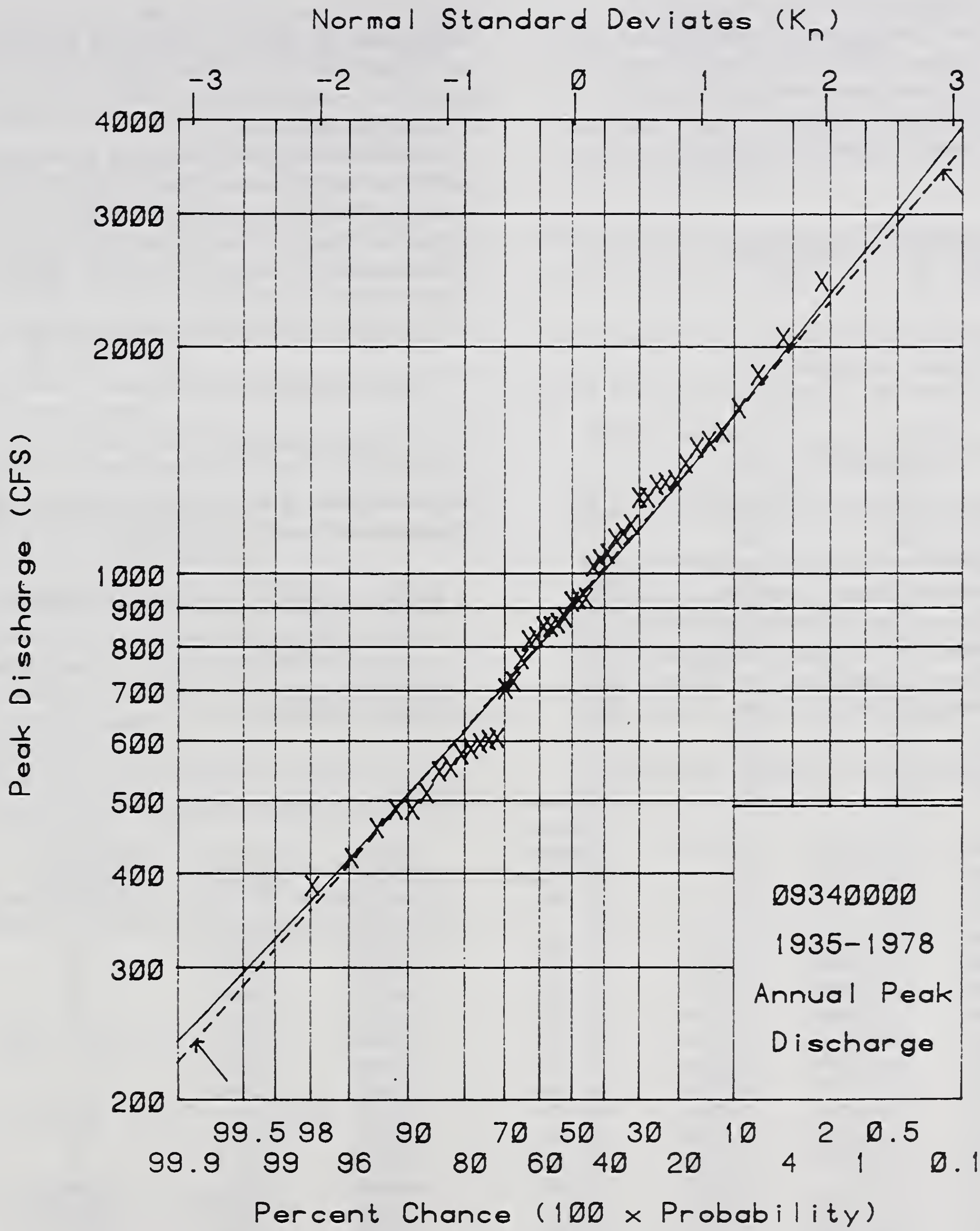

Figure 18-1.—Data and frequency curves for example 18-1. Solid line indicates log-normal distribution, and broken line indicates log-Pearson III. Arrows show values for outlier check.

Then compute differences between each sample logarithm and the mean logarithm, and use the sum of the squares and cubes of the differences (columns 4 and 5) in computing the standard deviation and skew.

Compute the standard deviation of logarithms by using the sum of squares of the differences and equation 2:

$$S = \left[\frac{1.659318}{(44 - 1)}\right]^{0.5} = 0.1964403$$

Compute the skew by using the sum of cubes of the differences (column 5) and equation 4:

$$G = \frac{44}{(44-1)\,(44-2)\,(0.1964403)^3} \times 0.023534 = 0.0756$$

For ease of use in next step, round skew value to the nearest tenth (G = 0.1).

4. Use SCS TR-38 to obtain K values for required skew at sufficient exceedance probabilities to define the frequency curve. Use the mean, standard deviation, skew, and equation 16 to compute discharges at the selected exceedance probabilities. The TR-38 K values and discharge computations are shown in table 18-4.

Plot the frequency curves on the same graph as the sample data (fig. 18-1). A comparison between the plotted frequency curve and the sample data verifies the selection of the distributions. Other distributions can be tested the same way.

5. Check the sample for outliers. K_n values, based on sample size, are obtained from exhibit 18-1. The K_n value for a sample of 44 is 2.945. Compute the log-normal high outlier criteria from the mean, the standard deviation, the outlier K value, and equation 16:

$$\log Q_{HI} = 2.957376 + (2.945)(0.1964403)$$

$$= 3.5359$$

$$Q_{HI} \doteq 3{,}435 \text{ cfs}$$

Use the negative of the outlier K_n value in equation 16 to compute the low outlier criteria:

$$\log Q_{LO} = 2.957376 + (-2.945)(0.1964403)$$

$$= 2.37886$$

$$Q_{LO} = 239 \text{ cfs}$$

Table 18-4.—Frequency curve solutions for example 18-1

Exceed. prob. (q)	TR-38 K value (G=0.0)	Log Q = $\bar{X}$+KS	Log-normal discharges (cfs)	TR-38 K value (G=0.1)	Log Q = $\bar{X}$+KS	Log-Pearson III discharges (cfs)
0.999	−3.09023	2.35033	224	−2.94834	2.37820	239
.998	−2.87816	2.39199	247	−2.75706	2.41578	260
.995	−2.57583	2.45138	283	−2.48187	2.46984	295
.99	−2.32635	2.50039	317	−2.25258	2.51488	327
.98	−2.05375	2.55394	358	−1.99973	2.56455	367
.96	−1.75069	2.61347	411	−1.71580	2.62032	417
.90	−1.28155	2.70563	508	−1.27037	2.70782	510
.80	−0.84162	2.79205	620	−0.84611	2.79117	618
.70	−0.52440	2.85436	715	−0.53624	2.85204	711
.60	−0.25335	2.90761	808	−0.26882	2.90457	803
.50	0.0	2.95738	907	−0.01662	2.95411	900
.40	0.25335	3.00714	1,017	0.23763	3.00406	1,009
.30	0.52440	3.06039	1,149	0.51207	3.05797	1,143
.20	0.84162	3.12270	1,326	0.83639	3.12168	1,323
.10	1.28155	3.20912	1,619	1.29178	3.21113	1,626
.04	1.75069	3.30128	2,001	1.78462	3.30795	2,032
.02	2.05375	3.36082	2,295	2.10697	3.37127	2,351
.01	2.32635	3.41436	2,596	2.39961	3.42876	2,684
.005	2.57583	3.46337	2,907	2.66965	3.48180	3,033
.002	2.87816	3.52276	3,332	2.99978	3.54665	3,521
.001	3.09023	3.56442	3,668	3.23322	3.59251	3,913

Because all of the sample data are between Q_{HI} and Q_{LO}, there are no outliers for the log-normal distribution.

High and low outlier criteria values for skewed distributions can be found by use of the high and low probability levels from exhibit 18-1. Read discharge values from the plotted log-Pearson III frequency curve at the probability levels listed for the sample size, in this case, 44. The high and low outlier criteria values are 3,700 and 250 cfs. Because all sample data are between these values, there are no outliers for the log-Pearson III distribution.

Example 18-2.—Development of a two-parameter gamma frequency curve.

Table 18-5 contains 7-day mean low flow data for the Patapsco River at Hollifield, Md. (Station 01589000), including the water year (column 1) and 7-day mean low-flow values (column 2). The remaining columns will be referenced in the following steps.

1. Plot the data. Before plotting, arrange the data in ascending order (column 3). Weibull plotting positions are computed based on the sample size of 34 from equation 8 (column 4). Ordered data are plotted at the computed plotting positions on logarithmic-normal probability paper (fig. 18-2).
2. Examine the trends of the plotted data. The data plot as a single trend with a slightly concave downward shape.
3. Compute the required statistics. Compute the gamma shape parameter, γ, from the sample data (column 3), equations 1, 9, and 10, and either equation 11 or 12.

$$\overline{X} = \frac{1876}{34} = 55.17647$$

$$G_m = (3.308266 \times 10^{55})^{1/34} = 42.94666$$

$$R = \ln\left[\frac{55.17647}{42.9466}\right] = 0.25058$$

Because R < 0.5772 use equation 11 to compute γ.

$$\gamma = (1/0.25058)\,\{0.5000876 + (0.1648852)(0.25058) - (0.0544274)(0.25058)^2\}$$

$$\gamma = 2.14697$$

Using the mean and γ, compute the standard deviation and skew from equations 13 and 14:

$$S = \frac{55.17647}{\sqrt{2.14697}} = 37.65658$$

$$G = \frac{2}{\sqrt{2.14697}} = 1.36495$$

For ease of use in next step, round skew value to the nearest tenth (G = 1.4).

Table 18-5.—Basic statistics data for example 18-2

Water year (1)	7-Day mean low flow (cfs) (2)	Ordered data (cfs) (3)	Weibull plot position 100 M/(N+1) (4)
1946	107	11	2.9
1947	127	15	5.7
1948	79	16	8.6
1949	145	17	11.4
1950	110	19	14.3
1951	98	20	17.1
1952	99	22	20.0
1953	168	23	22.9
1954	60	23	25.7
1955	20	25	28.6
1956	23	25	31.4
1957	51	25	34.3
1958	17	27	37.1
1959	52	32	40.0
1960	25	40	42.9
1961	43	43	45.7
1962	27	44	48.6
1963	16	47	51.4
1964	11	48	54.3
1965	19	50	57.1
1966	22	51	60.0
1967	15	52	62.9
1968	47	59	65.7
1969	32	60	68.6
1970	25	69	71.4
1971	25	79	74.3
1972	59	80	77.1
1973	69	98	80.0
1974	50	99	82.9
1975	44	107	85.7
1976	80	110	88.6
1977	40	127	91.4
1978	23	145	94.3
1979	48	168	97.1
Sum		1,876	
Product		3.308266×10^{55}	

4. Compute the frequency curve. Use TR-38 to obtain K values for the required skew at sufficient probability levels to define the frequency curve. Compute discharges at the selected probability levels (p) by equation 15. The TR-38 K values and computed discharges are shown in table 18-6. Then plot the frequency curve on the same graph as the sample data (fig. 18-2). Compare the plotted data and the frequency curve to verify the selection of the two-parameter gamma distribution.

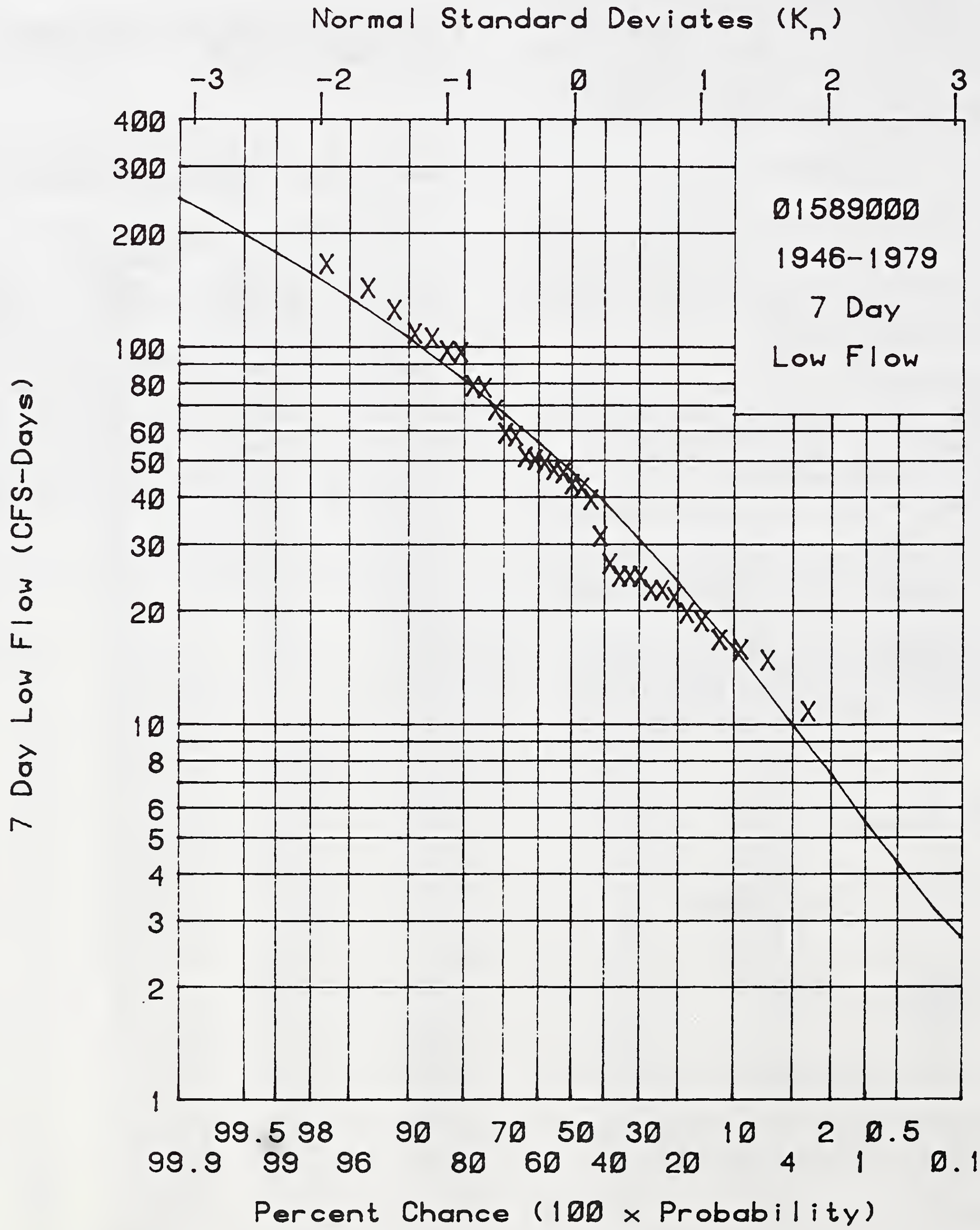

Figure 18-2.—Data and frequency curve for example 18-2.

Table 18-6.—Solution of frequency curve for example 18-2.

Prob. (p)	TR-38 K value (G=1.4)	Q = $\overline{X}$ + KS
0.999	5.09505	247.0
.998	4.55304	227.
.995	3.82798	199.
.99	3.27134	178.
.98	2.70556	157.
.96	2.12768	135.
.90	1.33665	106.
.80	0.70512	82.
.70	0.31307	67.
.60	0.01824	56.
.50	−0.22535	47.
.40	−0.43949	39.
.30	−0.63779	31.
.20	−0.83223	24.
.10	−1.04144	16.
.04	−1.19842	10.
.02	−1.26999	7.4
.01	−1.31815	5.5
.005	−1.35114	4.3
.002	−1.37981	3.2
.001	−1.39408	2.7

5. Check the sample for outliers. Obtain outlier probability levels from exhibit 18-1 for a sample size of 34. The probability levels are 0.9977863 and 0.0022137. From figure 18-2 read the discharge rates associated with these probability levels. The outlier criteria values are 220 and 3.3 cfs. Because all sample data are between these values, there are no outliers.

6. Use the frequency curve to estimate discharges at desired probability levels.

Example 18-3.—Development of a mixed distribution frequency curve by separating the data by cause (method 1) and by using at least the upper half of the data (method 2).

Method 1: The Causative Factor Method.—Annual peak discharge data for Carson River near Carson City, Nev., (Station 10311000) are given in table 18-7. Column 1 contains the water year, and column 2 contains annual peak discharge. The other columns will be referenced in the following steps:

1. Plot the data. Before plotting, order the data from high to low (column 3). Compute plotting positions, using sample size of 37 and equation 8 (column 4). Then plot ordered data at the computed plotting positions on logarithmic-normal probability paper (fig. 18-3).

2. Examine the plotted data. The data plot in an S-shape with a major trend break at 20-percent chance.

Table 18-7.—Annual peak discharge data for example 18-3

Water year (1)	Annual peak discharge (cfs) (2)	Ordered annual peaks (cfs) (3)	Weibull plotting position M/(N+1) (4)
1939	541	30,000	0.026
1940	2,300	21,900	.053
1941	2,430	15,500	.079
1942	5,300	8,740	.105
1943	8,500	8,500	.132
1944	1,530	5,300	.158
1945	3,860	4,430	.184
1946	1,930	4,190	.211
1947	1,950	3,860	.237
1948	1,870	3,750	.263
1949	2,420	3,480	.289
1950	2,160	3,480	.316
1951	15,500	3,330	.342
1952	3,750	3,180	.368
1953	1,900	3,100	.395
1954	1,970	2,430	.421
1955	1,410	2,420	.447
1956	30,000	2,300	.474
1957	1,900	2,260	.500
1958	3,100	2,160	.526
1959	1,690	1,990	.553
1960	1,100	1,970	.579
1961	808	1,950	.605
1962	1,950	1,950	.632
1963	21,900	1,930	.658
1964	1,160	1,900	.684
1965	8,740	1,870	.711
1966	1,280	1,690	.737
1967	4,430	1,530	.763
1968	1,390	1,410	.789
1969	4,190	1,390	.816
1970	3,480	1,330	.842
1971	2,260	1,280	.868
1972	1,330	1,160	.895
1973	3,330	1,100	.921
1974	3,180	808	.947
1975	3,480	541	.974

3. Determine what caused the peak discharge. Based on streamgage and weather records, two causative factors were rainfall and snowmelt. Annual peak discharge series for each cause are tabulated in table 18-8.

4. Plot each annual series. As in step 1, arrange the data in descending order (rainfall, column 4; snowmelt, column 5) and compute plotting positions (column 6). Rainfall data are plotted in figure 18-4, and snowmelt data are plotted in figure 18-5.

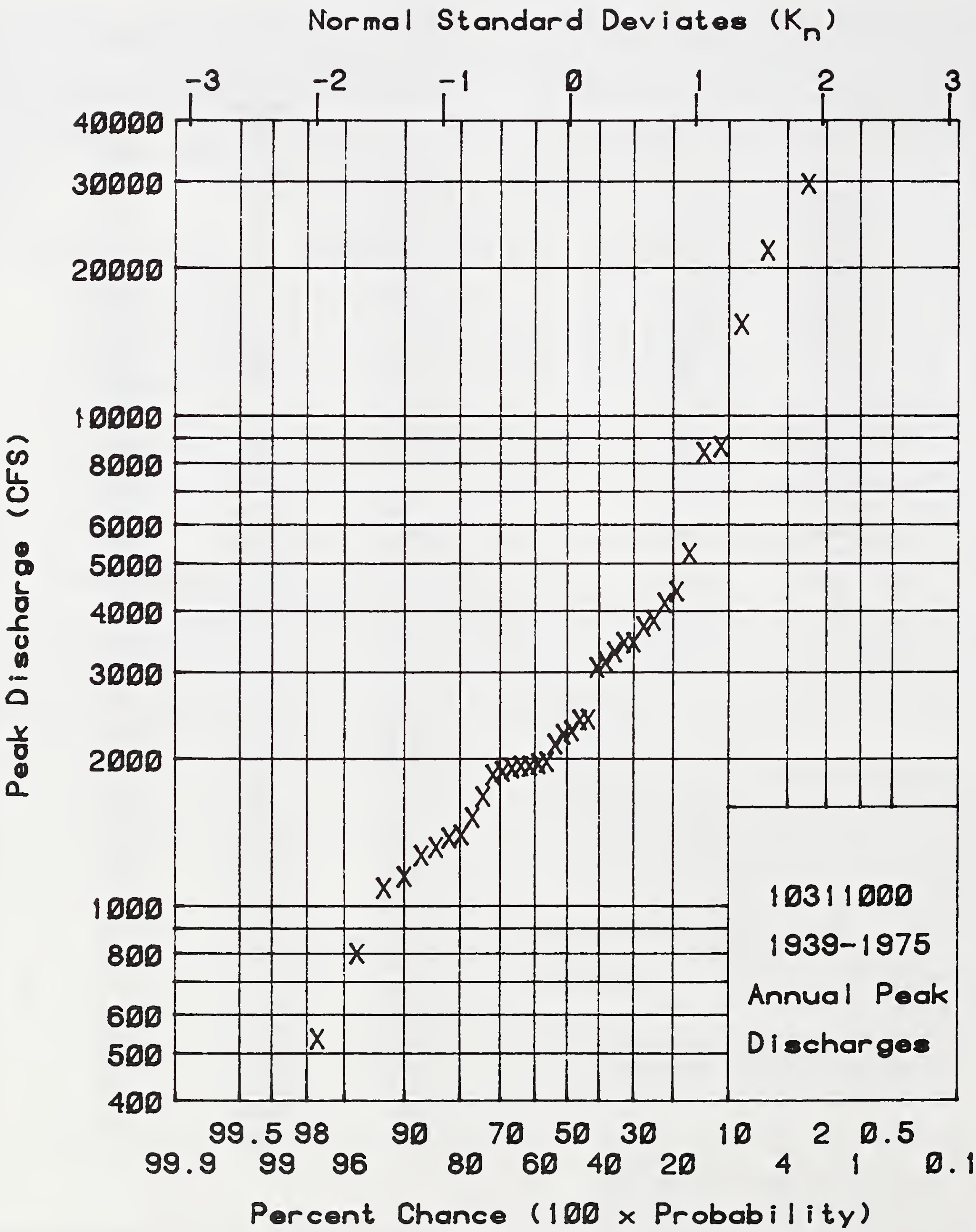

Figure 18-3.—Annual peak discharge data for example 18-3.

Table 18-8.—Annual rainfall/snowmelt peak discharge for example 18-3

Water year (1)	Annual rainfall peak discharge (cfs) (2)	Annual snowmelt peak discharge (cfs) (3)	Ordered rainfall peak discharge (cfs) (4)	Ordered Snowmelt peak discharge (cfs) (5)	Weibull plot position M/(N+1) (6)
1939	541	355	30,000	4,290	0.026
1940	1,770	2,300	21,900	4,190	.053
1941	1,015	2,434	15,500	3,480	.079
1942	5,298	2,536	8,740	3,330	.105
1943	8,500	2,340	8,500	3,220	.132
1944	995	1,530	5,298	3,100	.158
1945	3,860	1,420	4,430	2,980	.184
1946	1,257	1,930	3,860	2,759	.211
1947	1,950	1,680	3,750	2,536	.237
1948	755	1,870	3,560	2,460	.263
1949	2,420	1,600	3,480	2,434	.289
1950	1,760	2,158	2,172	2,417	.316
1951	15,500	1,750	2,946	2,340	.342
1952	3,750	2,980	2,590	2,300	.368
1953	1,990	972	2,420	2,158	.395
1954	1,970	1,640	2,260	2,010	.421
1955	1,410	1,360	2,120	1,930	.447
1956	30,000	3,220	1,990	1,900	.474
1957	1,860	1,900	1,970	1,900	.500
1958	2,120	3,100	1,950	1,870	.526
1959	1,690	698	1,950	1,750	.553
1960	1,090	895	1,860	1,680	.579
1961	814	620	1,770	1,680	.605
1962	1,950	1,900	1,760	1,640	.632
1963	21,900	2,417	1,690	1,530	.658
1964	1,160	800	1,410	1,420	.684
1965	8,740	2,460	1,257	1,360	.711
1966	920	1,280	1,160	1,360	.737
1967	4,430	4,290	1,090	1,309	.763
1968	936	1,360	1,015	1,280	.789
1969	3,560	4,190	995	972	.816
1970	3,480	2,010	975	895	.842
1971	2,260	837	936	837	.868
1972	975	1,309	920	800	.895
1973	2,946	3,330	814	698	.921
1974	3,172	2,759	755	620	.947
1975	2,590	3,480	541	355	.974

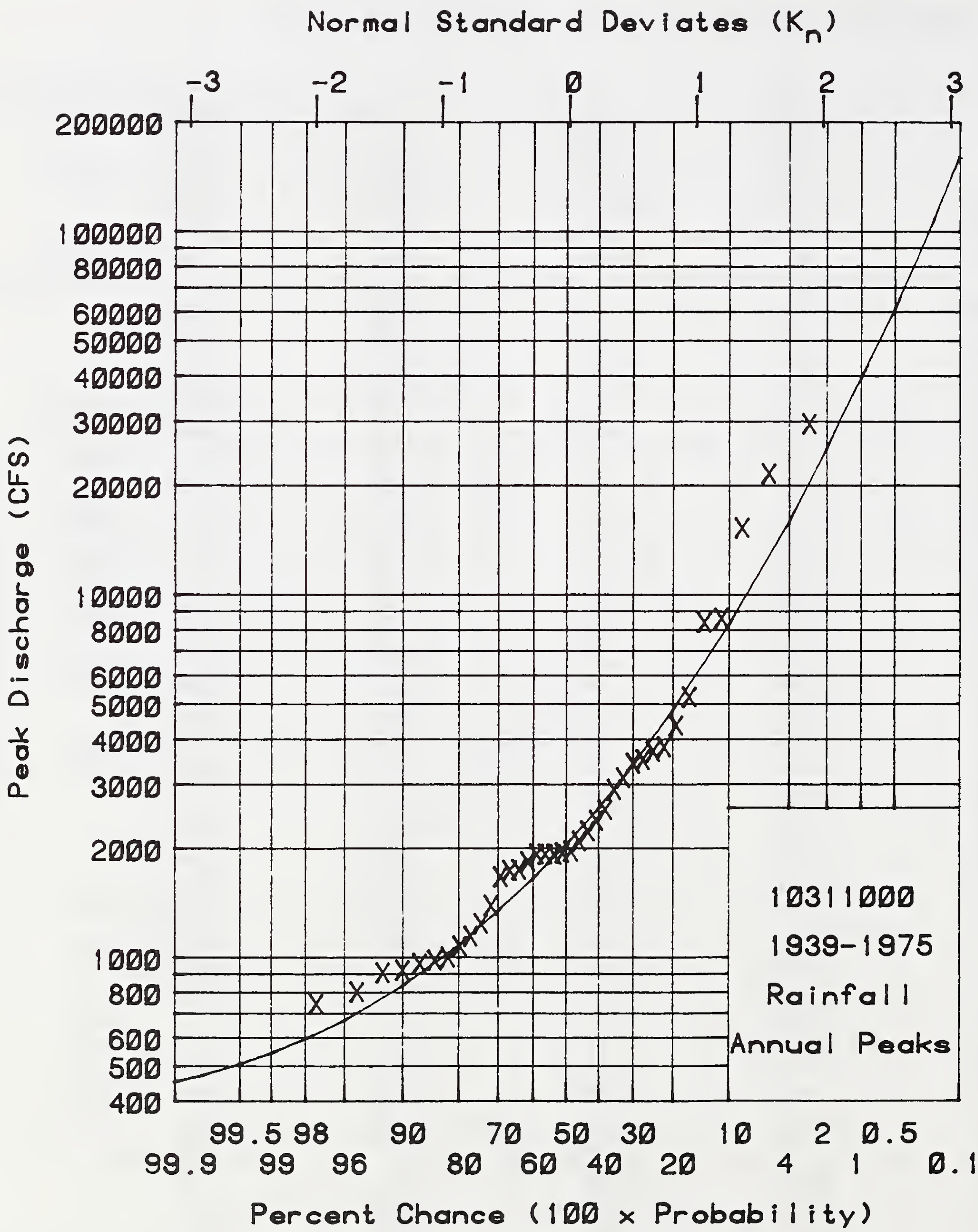

Figure 18-4.—Data and frequency curve for example 18-3.

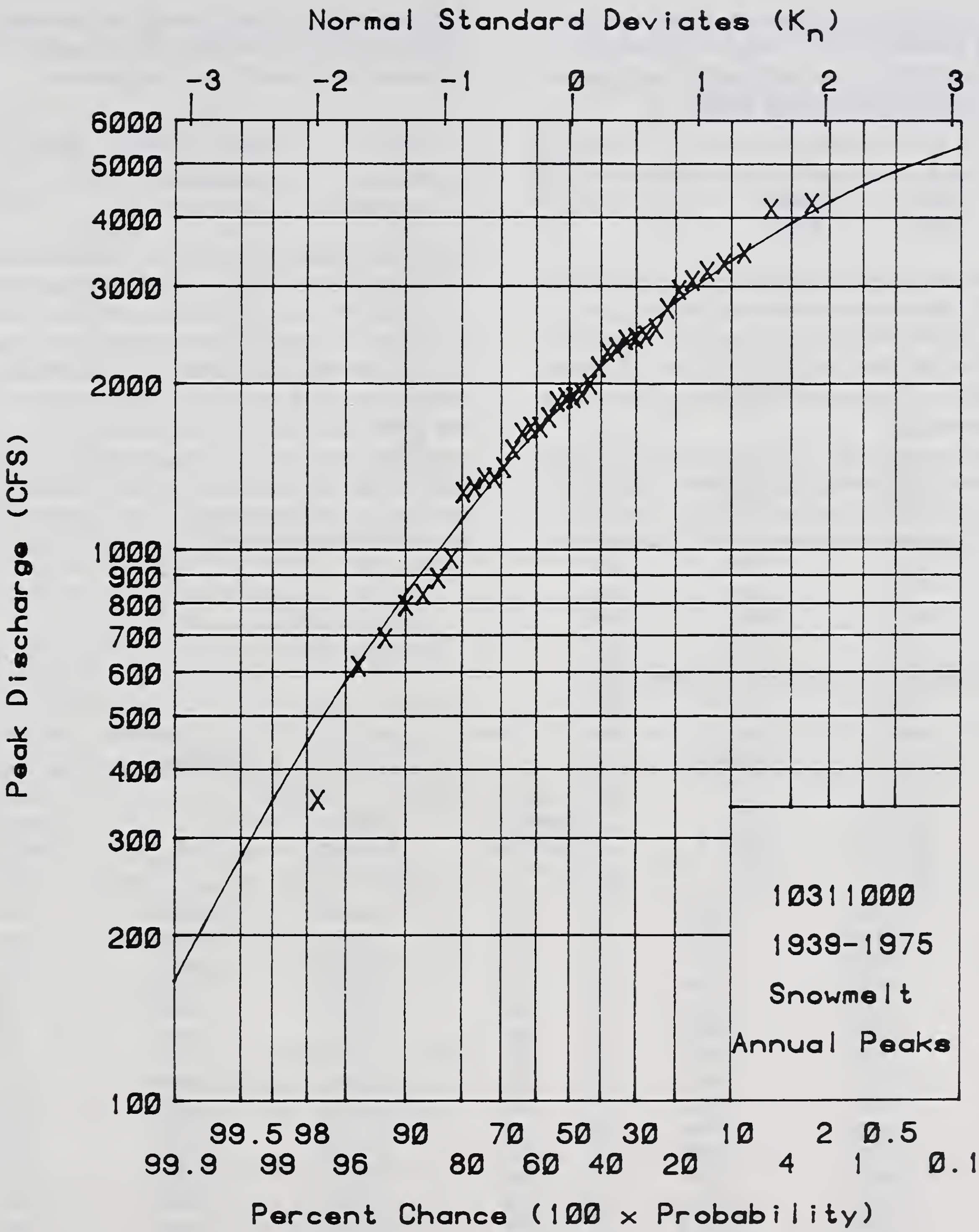

Figure 18-5—Data and frequency curve for example 18-3.

5. Compute the required statistics. Using the procedure in step 3 of example 18-1, compute the sample mean, standard deviation, and skewness for each series. The results of these computations follow:

Series	$\overline{X}$	*S*	*G*	*Use G*
Rainfall	3.37611	0.40385	1.03	1.0
Snowmelt	3.24241	0.24176	− 0.77	− 0.8

6. Compute the log-Pearson III frequency curve for each series. The frequency curve solution for each series, as computed in step 4 of example 18-1, is listed in table 18-9. Log-Pearson frequency curves are plotted as for the rainfall and the snowmelt series in figures 18-4 and 18-5, respectively.

7. Check each sample for outliers. Read high and low outlier criterion values from the frequency curve plots (figures 18-4 and 18-5) at the probability levels given in exhibit 18-1 for the sample size of 37. The high and low probability levels from exhibit 18-1 are 0.9980116 (99.8 percent) and 0.0019884 (0.2 percent). The outlier criterion values read from the plots are:

Series	*High Outlier Criterion*	*Low Outlier Criterion*
Rainfall	106,000 cfs	470 cfs
Snowmelt	5,200 cfs	200 cfs

All of the rainfall and snowmelt data are between the outlier criterion values, so there are no outliers.

8. Combine the rainfall and snowmelt series frequency curves. For selected discharge values, read the rainfall and snowmelt frequency curve probability levels from figures 18-4 and 18-5. Using equation 17, combine the probabilities for the two series. Table 18-10 contains the individual and combined probabilities of selected discharges. The snowmelt frequency curve approaches an upper bound of 5,400 cfs; therefore, only the rainfall curve is used above this value.

Table 18-9.—Frequency curve solutions for example 18-3

	Rainfall			Snowmelt		
Exceed. prob. (q)	TR-38 K value (G = 1.0)	Log Q = $\overline{X}$ + KS	Log-Pearson III discharges (cfs)	TR-38 K value (G = −0.8)	Log Q = $\overline{X}$ + KS	Log-Pearson III discharges (cfs)
0.999	−1.78572	2.65495	452	−4.24439	2.21629	165
.998	−1.74062	2.67316	471	−3.84981	2.31168	205
.995	−1.66390	2.70414	506	−3.31243	2.44160	276
.99	−1.53838	2.73464	543	−2.89101	2.54348	350
.98	−1.49188	2.77361	594	−2.45298	2.64938	446
.96	−1.36584	2.82452	668	−1.99311	2.76056	576
.90	−1.12762	2.92072	833	−1.33640	2.91932	830
.80	−0.85161	3.03219	1,077	−0.77986	3.05387	1,132
.70	−0.61815	3.12816	1,343	−0.41309	3.14254	1,388
.60	−0.39434	3.21686	1,648	−0.12199	3.21292	1,633
.50	−0.16397	3.30989	2,041	0.13199	3.27432	1,881
.40	0.08763	3.41150	2,579	0.36889	3.33159	2,146
.30	0.38111	3.53002	3,389	0.60412	3.38846	2,446
.20	0.75752	3.68203	4,809	0.85607	3.44937	2,814
.10	1.34039	3.91743	8,268	1.16574	3.52424	3,344
.04	2.04269	4.20105	15,887	1.44813	3.59251	3,913
.02	2.54206	4.40272	25,277	1.60604	3.63069	4,273
.01	3.02256	4.59677	39,516	1.73271	3.66131	4,585
.005	3.48874	4.78504	60,959	1.83660	3.68643	4,858
.002	4.08802	5.02706	106,428	1.94806	3.71337	5,169
.001	4.53112	5.20600	160,695	2.01739	3.73013	5,372

Table 18-10.—Combination of frequency curves for example 18-3

Peak discharge (cfs)	P_R = P (rain)	P_S = P (snow)	P = $P_R + P_S - P_R P_S$
600	0.98	0.955	0.999
830	.90	.90	.990
1,640	.60	.60	.840
2,450	.34	.30	.538
3,360	.30	.10	.370
4,840	.20	.005	.204
8,180	.10	—[1]	.100
16,030	.04	—	.040
41,360	.01	—	.010
180,560	.001	—	.001

[1] Probability is too small to be considered.

9. Figure 18-6 shows the combined and annual frequency curves plotted on the same sheet as the annual series. The combined series frequency curve will not necessarily fit the annual series, as additional data were used to develop it, but the curve does represent the combined effect of the two causes.

Method 2: Truncated series.—An alternative method of mixed distribution analysis is to fit a log-normal distribution to only part of the data. At least the upper half of the data must be included and must be basically log-normal (i.e., approximate a straight line when plotted on logarithmic-normal paper). Steps 1 and 2, method 1, help to determine that the data are mixed and that the major trend break occurs at 20 percent. While the upper half of the data will include data from both major trends, a log-normal fit will be used as an illustration of the procedure.

1 and 2. See method 1.

3. Select the normal K values for a sample size of 37 from exhibit 18-2. A tabulation of these values along with the ordered annual peaks and their logarithms is in table 18-11.

4. Plot the data. Plot the ordered annual peaks at the normal K values tabulated in table 18-11. These are plotted in figure 18-7. For plotting the data, use the normal K-value scale at the top of the figure.

5. Compute the statistics based on the upper half of data. Use equation 18 and 19 to compute the mean and standard deviation from the sums given in table 18-11.

$$S = \left[\frac{260.757 - (70.11699)^2/19}{17.25002 - (14.44423)^2/19}\right]^{0.5} = 0.56475$$

$$\overline{X} = [70.11699 - (0.56475)(14.44423)]/19 = 3.26103$$

6. Compute the log-normal frequency curve for the data. Use the same procedure as explained in step 4 of example 18-1. As a log-normal curve is to be fit and it will be a straight line on logarithmic-normal paper, solution of only two points is required.

Table 18-11.—Data and normal K values for example 18-3

Ordered annual peaks (cfs) (1)	Logarithm of ordered peak (2)	Expected normal K value (3)	Expected normal K value (4)
30,000	4.47712	2.12928	
21,900	4.34044	1.71659	
15,500	4.19033	1.47676	
8,740	3.94151	1.30016	
8,500	3.92942	1.15677	
5,300	3.72428	1.03390	
4,430	3.64640	0.92496	
4,190	3.62221	0.82605	
3,860	3.58659	0.73465	
3,750	3.57403	0.64902	
3,480	3.54158	0.56793	
3,480	3.54158	0.49042	
3,330	3.52244	0.41576	
3,180	3.50243	0.34336	
3,100	3.49136	0.27272	
2,430	3.38561	0.20342	
2,420	3.38382	0.13509	
2,300	3.36173	0.06739	
2,260	3.35411	0.00000	
2,160			−0.06739
1,990			−0.13509
1,970			−0.20342
1,950			−0.27272
1,950			−0.34336
1,930			−0.41576
1,900			−0.49042
1,870			−0.56793
1,690			−0.64902
1,530			−0.73465
1,410			−0.82605
1,390			−0.92496
1,330			−1.03390
1,280			−1.15677
1,160			−1.30016
1,100			−1.47676
808			−1.71659
541			−2.12928
Sum (values)	70.11699	14.44423	
Sum (values²)	260.75700	17.25002	

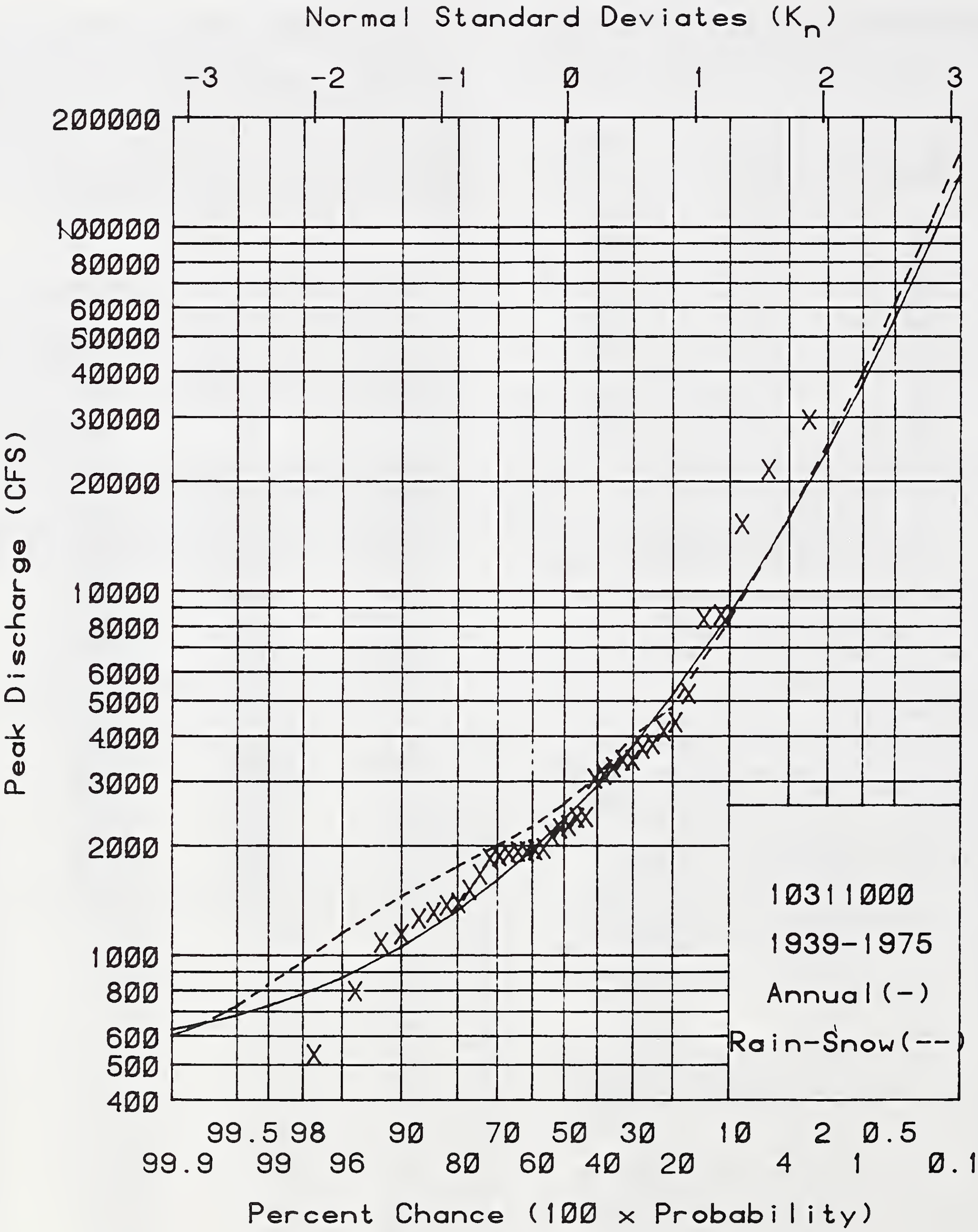

Figure 18-6.—Annual and rain-snow frequency curves for example 18-3.

Probability level	*K* normal	*Log Q =* $\overline{X}$ *+ KS*	Q *(cfs)*
0.50	0.0	3.26103	1,824
0.01	2.32635	4.57484	37,570

7. Plot the computed frequency curve. The curve is plotted on the same page as the sample data, figure 18-7.

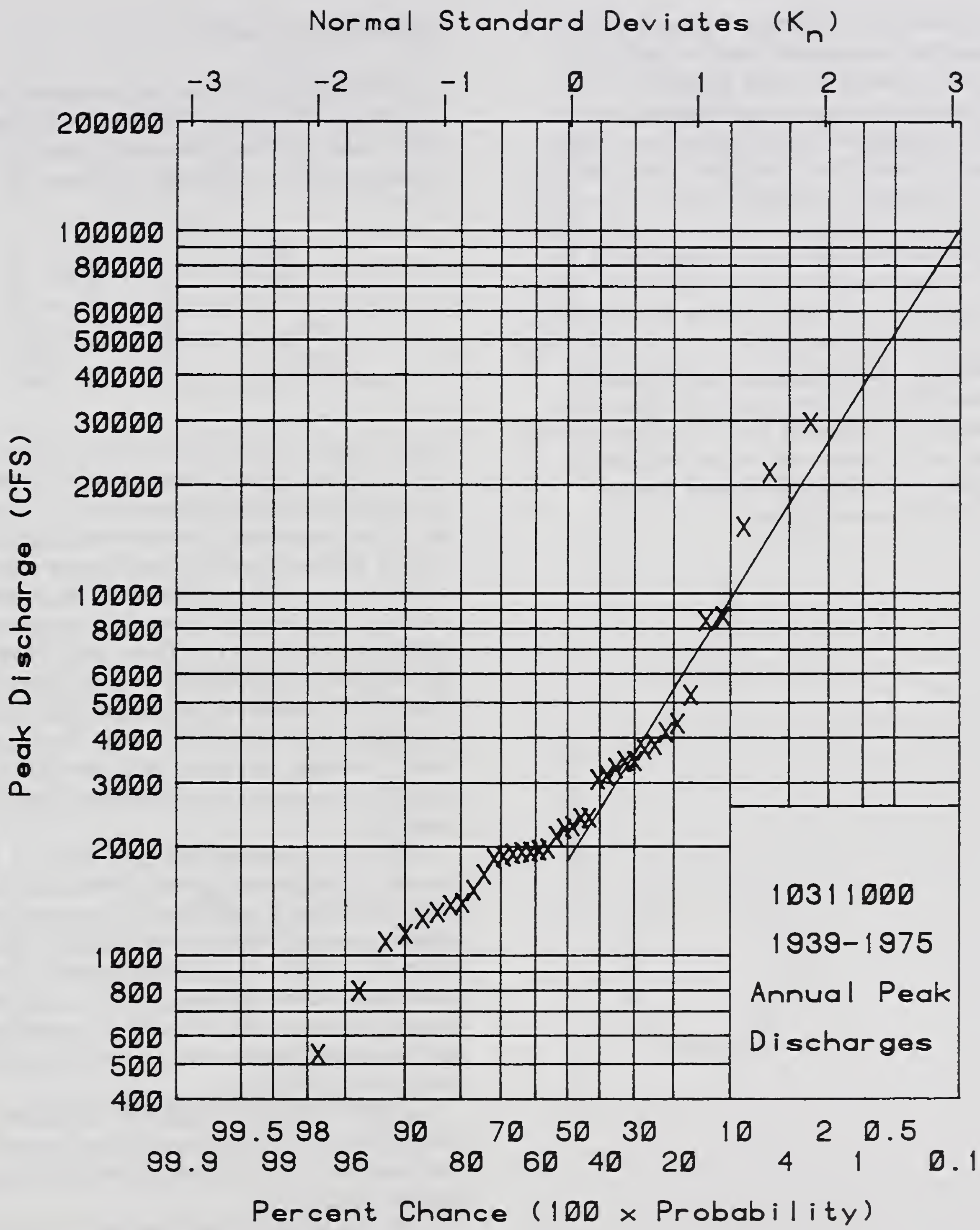

Figure 18-7.—Data and top half frequency curve for example 18-3.

A flow duration curve indicates the percentage of time a streamflow was greater than or less than a specific discharge during a period of record. A flow duration curve does not show the chronological sequence of flows. Because daily flows are nonrandom and nonhomogeneous, a flow duration curve cannot be considered a frequency or probability curve. Duration curves are normally constructed from mean daily flows.

Although a flow duration curve indicates only the distribution of mean daily flows that have been recorded, it can be used as an estimate of the flow duration distribution expected. Flow duration curves help determine availability of streamflow for beneficial uses.

USGS Water Supply Paper 1542-A (Searcy 1959) gives procedures for preparing and using flow duration curves. Many flow duration curves are available in USGS publications. Unpublished curves may be available at USGS District Offices.

Correlation Analysis

Correlation is an index that measures the linear variation between variables. While several correlation coefficients exist, the most frequently used is the *Pearson product-moment correlation coefficient* (r):

$$r = \frac{\sum_{i=1}^{N} (X_i - \overline{X})(Y_i - \overline{Y})}{\left[\sum_{i=1}^{N} (X_i - \overline{X})^2 \sum_{i=1}^{N} (Y_i - \overline{Y})^2\right]^{0.5}} \qquad (18\text{-}20)$$

where X_i and Y_i are values of the i^{th} observation of the two variables X and Y, respectively; $\overline{X}$ and $\overline{Y}$ are the means of the two samples; and N is the number of common elements in the samples. Equation 20 is used to measure the relationship between two variables. As an example, one may be interested in examining whether or not there is a significant linear relationship between the T-year peak discharge (Y) and the fraction of the drainage area in impervious land cover (X). To examine this relationship, one would need to obtain values for X and Y from N watersheds with widely different values of the X variable, and use equation 20 to determine a quantitative index of the relationship.

Values of r range between +1 and −1. A correlation of +1 indicates a perfect direct relationship between variables X and Y, while a correlation of −1 indicates a perfect inverse relationship. Zero correlation indicates no linear relationship between the variables. Correlation values between 0 and ±1 indicate the degree of relationship between the variables. Figure 18-8 illustrates various linear correlation values between two variables.

Because correlation coefficient values can be misleading at times, the sample data should be plotted and examined. Some situations that may cause low correlation values are:

1. No relationship exists between variables—random variation.
2. A relationship exists but is nonlinear, such as a parabolic or circular relationship.
3. Data values can depart significantly from the linear trend of the remaining data. The extreme values not only can change the correlation coefficient, but also can change the sign of the correlation coefficient.

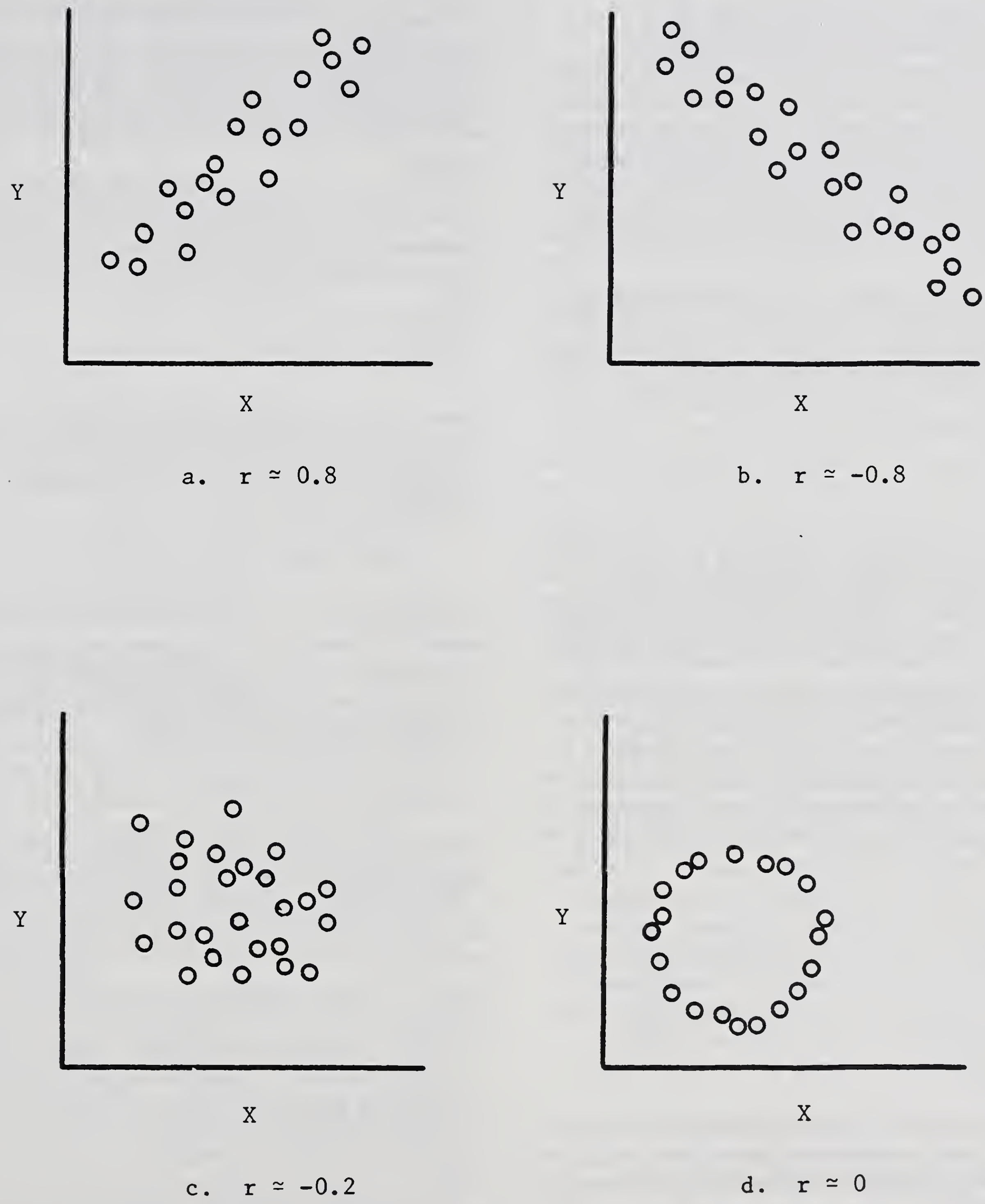

Figure 18-8.—Linear correlation values.

High correlation can be attributed to:

1. Significant relationship between variables.

2. Small sample size—for example, two points defining a straight line will result in a correlation coefficient of r = 1 or −1. Other small samples are influenced by this effect and may also have high correlation values.

3. Data clustering—two data clusters, each with low correlation, can exhibit high correlation values. Each cluster acting as a unit value may act as a small sample size.

The correlation between two variables will change if either of the variables is transformed nonlinearly. A new correlation coefficient should be developed for the transformed variables and will apply only to the variables in their transformed state.

Regression

Regression is a method of developing a relationship between a *criterion* variable (Y) and one or more *predictor* variables (X), with the objective of predicting the criterion variable for given values of the predictor variables.

Correlation analysis is quite different from regression analysis, although they are frequently used together. Regression is a predictive technique that distinguishes between the predictor and criterion variables. A regression equation that is developed to predict Y should not be transformed to predict the X variable for a given value of Y. Regression is based on an assumption that no error exists in the independent variable; errors occur only in the dependent variable. Thus, regression is directional. Correlation is not directional in that the correlation between Y and X is the same as that between X and Y. Also, correlation is different from regression in that correlation is only a standardized index of the degree of a linear relationship.

Wang and Huber (1967) list additional assumptions that form the basis for regression as:

1. The predictor variables are statistically independent.

2. The variance of the criterion variable does not change with changes in magnitude of the predictor variables.

3. The observed values of the criterion variable are uncorrelated events.

4. The population of the criterion variable is normally distributed about the regression line for any fixed level of the predictor variables under consideration.

Generally, hydrologic data do not meet all of the assumptions of regression analysis, but regression is still used because it provides an easy method for analyzing many factors simultaneously. The error caused by failure to meet all of the assumptions is generally minor.

There are several forms of regression analysis, including linear bivariate, linear multiple, and curvilinear. The *linear bivariate regression* relates a criterion variable (Y) and a single predictor variable (X) by using:

$$Y = a + bX \tag{18-21}$$

where a and b are the intercept and slope regression coefficients, respectively. *Linear multiple regression* relates a criterion variable (Y) and p predictor variables (X_j where j = 1, 2, . . ., p):

$$Y = b_0 + b_1X_1 + b_2X_2 \ldots + b_pX_p \tag{18-22}$$

where b_j(j = 0, 1,. . . , p) are the partial regression coefficients. The *curvilinear regression* technique is used when powers of the predictor variable(s) are included in the equation. For a single variable the following regression equation can be used:

$$Y = b_0 + b_1X + b_2X^2 + \ldots + b_qX^q \tag{18-23}$$

where q is the order of the polynomial. This equation can be expanded to include other predictor variables.

More than one regression equation can be derived to fit data, so some technique must be selected to evaluate the "best fit" line. The *method of least squares* is usually used because it minimizes the sum of the square of the differences between the sample criterion values and the estimated criterion values.

A cause-and-effect relationship is implied between the predictor and the criterion variables. If there is no physical relationship between a predictor and the criterion, do not use the predictor. Always carefully examine the sign of the coefficients for rationality. Do not use any equation outside the range of the sample data that were used to derive the coefficients.

A detailed procedure of how to develop regression equations is not given in this chapter. Regression analysis is usually performed by use of programmed procedures on a calculator or computer. The following discussion highlights the basic concepts and terminology of regression analysis.

Evaluating Regression Equations

After the regression coefficients are developed, it is necessary to examine the quality of a regression equation. The following means of evaluating the quality are discussed:

1. Analysis of the residuals.
2. The standard error of estimate.
3. The coefficient of determination.
4. Analysis of the rationality of the sign and magnitude of the regression coefficients.
5. Analysis of the relative importance of the predictor variables, as measured by the standardized partial regression coefficients.

A *residual* is the difference between the value predicted with the regression equation and the criterion variable. Residuals measure the amount of criterion variation left unexplained by the regression equation. The least squares concept assumes that the residuals should exhibit the following properties:

1. Their mean value equals zero.
2. They are independent of criterion and predictor variables.
3. Their variance is constant.
4. They have a normal distribution.

The mean of zero is easily verified by simply summing the residuals; a nonzero mean may result if not enough digits are used in the partial regression coefficients. Their independence and constant variance can be checked by plotting the residuals against the criterion and each predictor. Such plots should not exhibit any noticeable trends. Figure 18-9 illustrates some general trends that might occur when residuals are plotted. Nonconstant variance usually indicates an incorrect model form.

In theory the residuals are normally distributed. The distribution can often be identified by use of a frequency analysis. However, if the sample is small, it is difficult to make conclusive statements about the distribution of the residuals. Frequently, the model can be improved if a cause for a residual or trends in residuals are found.

Just as the individual residuals are of interest, the moments of the residuals are also worth examining. While the mean of the residuals is 0, the standard deviation of the residuals is called the *standard error of estimate*, which is denoted by S_e and is computed by:

$$S_e = \left[\frac{\sum_{i=1}^{N} (\hat{Y}_i - Y_i)^2}{d_f} \right]^{0.5} \tag{18-24}$$

where $\hat{Y}_i$ is the predicted value and Y_i the observed value of the i^{th} observation on the criterion variable, and d_f is the degrees of freedom. The *degrees of freedom* equal the number of independent pieces of information required to form the estimate. For a regression equation this equals the number of observations in the data sample minus the number of unknowns estimated from the data. A regression equation with p predictor variables and an intercept coefficient would have $N - p - 1$ degrees of freedom.

Compare S_e with the standard deviation of the criterion variable (S_y) as a measure of the quality of a regression equation. Both S_e and S_y have the same units as the criterion variable. If the regression equation does not provide a good fit to the observed values of the criterion variable, then S_e should approach S_y, with allowance being made for the differences in degrees of freedom (S_e has $N - p - 1$ while S_y has $N - 1$). However, if the regression provides a good fit, S_e will approach zero. Thus, S_e can be compared with the two extremes, 0 and S_y to assess the quality of the regression.

It is important to consider the portion of the total variation in the criterion variable that is explained by the regression equation. The explained portion is called the *coefficient of determination* and can be computed by:

$$r^2 = \frac{\sum_{i=1}^{N} (\hat{Y}_i - \overline{Y})^2}{\sum_{i=1}^{N} (Y_i - \overline{Y})^2} \tag{18-25}$$

The value of r^2 ranges from 0 to 1, with a value of 0 indicating no relationship between the criterion and predictor variables and a value of 1 indicating a perfect fit of the sample data to the regression line. The value of r^2 is a decimal percentage of the variation in Y explained by the regression equation.

There is an inverse relationship between r^2 and S_e. Some texts gives the relationship as:

$$S_e = S\sqrt{1 - r^2} \tag{18-26}$$

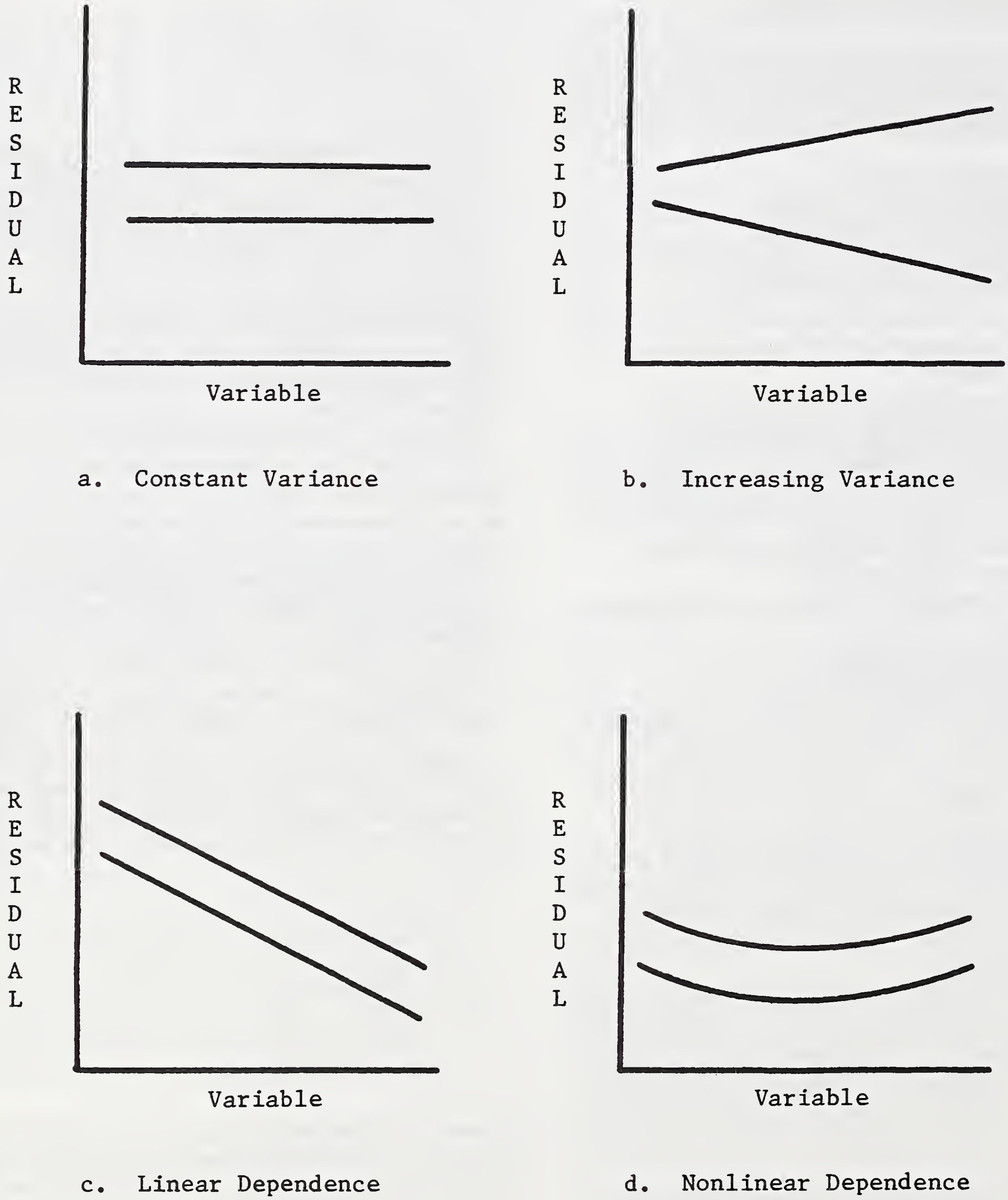

Figure 18-9.—Sample plots of residuals.

While this relationship may be acceptable for large samples, it should not be used for small samples because S_e is based on $N - p - 1$ degrees of freedom, while S is based on $N - 1$ degrees of freedom and r^2 is based on N degrees of freedom. Therefore, equation 24 and 25 should be used to compute S_e and r^2.

A regression equation describes the relationship that exists between the variables, with a partial regression coefficient reflecting the effect of the corresponding predictor variable on the criterion variable. As such, the magnitude and sign of each coefficient should be checked for rationality. While it is sometimes difficult to assess the rationality of the magnitude of a coefficient, it is usually easy to assess the rationality of the sign of the coefficient. Irrationality of either sign or magnitude often results from significant correlations between predictor variables. Thus, the use of highly correlated predictor variables should be avoided. The potential accuracy of estimates is rarely increased significantly by including a predictor variable that is highly correlated with one or more other predictor variables in the equation.

Regression equations can be developed for any number of predictor variables, but selecting the proper number is important. Having too few predictor variables may reduce the accuracy of the criterion estimate. Having too many predictor variables makes the equation more complex than necessary and wastes time and money in collecting and processing unneeded data that do not significantly improve accuracy.

Step-type regressions can be used to evaluate the importance (significance) of individual predictor variables in a regression equation. A step consists of adding or deleting a predictor variable from the regression equation and measuring the increase or decrease in the ability of the equation to predict the criterion variable.

The significance of predictor variables and the total equation are evaluated by using F-tests. Two F-tests are used, the *partial F-test* (F_p) checks the significance of predictor variables that are added or deleted from a regression equation while the *total F-test* (F_t) checks the significance of the entire regression equation. The partial F-test (F_p) is computed by:

$$F_p = \frac{(r_p^2 - r_{p-1}^2)}{(1 - r_p^2)/(N - p - 1)} \tag{18-27}$$

where r_p and r_{p-1} are the coefficients of determination for the p and p − 1 predictor models.

The equation is significant if the computed F is greater than the value found in an F distribution table. The degrees of freedom needed for use of the F table are $1(d_{f1})$ and $N - p - 1(d_{f2})$. F distribution tables for 0.05 and 0.01 levels of significance can be found in most standard statistics texts. The 0.05 probability table is generally used.

F_t is computed by:

$$F_t = \frac{r_p^2/p}{(1 - r_p^2)/(N - p - 1)} \tag{18-28}$$

where p is the number of predictors in the equation and r_p^2 is the coefficient of determination for the p predictor equation. The degrees of freedom required to use the tables are $p(d_{f1})$ and $N - p - 1(d_{f2})$.

Step backward regression starts with all predictors in the regression equation. The least important predictor is deleted and the F_p computed. If the predictor is not significant, the next least important of the remaining predictors is deleted and the process repeated. When a significant predictor is found, the previous equation that includes that predictor should be used.

Step forward regression starts with the most important predictor as the only variable in the equation. The most important of the remaining predictors is added and the F_p computed. If this predictor is significant, the next most important of the remaining predictors is added and the process repeated. When a nonsignificant predictor is found, the previous equation that does not include that predictor should be used.

Stepwise regression combines features of both step backward and step forward regression. Stepwise is basically a step forward regression with a step backward partial F test of all predictors in the equation after each step. When predictors are added to an equation, two or more may combine their prediction ability to make previously included predictors insignificant. As these "older" predictors are no longer needed in the equation, they are deleted.

Outline of Procedures

Correlation

1. Determine that a cause-and-effect relationship exists for all variable pairs to be tested.
2. Plot every combination of one variable vs. another to examine data trends.
3. (Optional.) make adjustments such as transformation of data if required.

4. Compute linear correlation coefficients between each pair of variables.

Regression

1. Compile a list of predictor variables that are related to the criterion variable by some physical relationship and for which data are available.
2. Plot each predictor variable versus the criterion variable.
3. Determine the form of the desired equation, i.e., linear or curvilinear.
4. Compute the correlation matrix, i.e., the correlation coefficient between each pair of variables.
5. Compute the regression coefficients for the predictor variable(s) that have high correlation coefficients with the criterion variable and low correlation coefficients with any other included predictor variables.
6. Compute standard error of estimate, S_e; standard deviation of the criterion variable, S_y; and the coefficient of determination, r^2.
7. Evaluate the regression equation by the following methods:
   a. Standard error of estimate has the bounds $0 \leq S_e \leq S_y$; as $S_e \rightarrow 0$ more of the variance is explained by the regression.
   b. Coefficient of determination has the bounds $0 \leq r^2 \leq 1$; as $r^2 \rightarrow 1$ the better the "fit" is of the regression line to the data.
   c. Partial and total F-tests are used to evaluate each predictor and total equation significance.
   d. The sign of each regression coefficient should be compared to the correlation coefficient for the appropriate predictor criterion. The signs should be the same.
   e. Examine the residuals to identify deficiencies in the regression equation and check the assumptions of the model.
8. If regression equation accuracy is not acceptable, reformulate the regression equation or transform some of the variables. A satisfactory solution is not always possible from data available.

Example 18–4.—Development of a multiple regression equation.

Peak flow data for watershed W-11, Hastings, Nebr., are used. Table 18-12 contains basic data for peak flow and three other variables. Use the following steps to develop the regression equation:

1. Plot one variable vs. another to establish that a linear or nonlinear data trend exists. Figure 18-10 is a plot of peak flow (Y) vs. maximum average 1-day flow (X_1). Similar plots are done for all combinations of variable pairs. The plot indicates a linear trend exists between peak flow and maximum average 1-day flow.
2. Determine the linear correlation coefficients between each pair of variables. Table 18-12 contains the product of differences required for the computation. Use equation 20 to compute the linear correlation. The array of the computed linear correlations follows:

Linear Correlation Matrix

	$q =$ Y	$Q =$ X_1	$Q_m =$ X_2	$P_m =$ X_3
Y	1.0000	0.9230	0.7973	0.5748
X_1		1.0000	0.9148	0.7442
X_2			1.0000	0.8611
X_3				1.0000

3. Develop a multiple regression equation based on maximum 1-day flow (X_1) and maximum monthly rainfall (X_3). Maximum monthly runoff (X_2) is not included as a predictor because it is highly correlated (0.9148) with maximum average 1-day flow (X_1). Predictor variables should be correlated with the criterion but not highly correlated with the other predictors. Two highly correlated predictors will explain basically the same part of the criterion variation. The predictor with the highest criterion correlation is retained. High correlation between predictor variables may cause irrational regression coefficients.

The following regression coefficients were developed from a locally available multiple linear regression computer program (Dixon 1975):

$b_0 = 0.0569$
$b_1 = 0.1867$
$b_2 = -0.0140$

The regression equation is:

$$Y = 0.0569 + 0.1867X_1 - 0.0140X_3$$

Table 18-12. – Basic correlation data for example 18-4 (Linear correlation coefficient computation)

Water year	Y = Peak flow (in/hr)	X_1 = Max. avg. 1-day flow (in)	X_2 = Max. month. runoff (in)	X_3 = Max. month. rainfall (in)	$(X - \overline{X})$ for Y	$(X - \overline{X})$ for X_1	$(X - \overline{X})$ for X_2	$(X - \overline{X})$ for X_3	Product of differences for Y,X_1	Y,X_2	Y,X_3	X_1,X_2	X_1,X_3	X_2,X_3
1939	0.0100	0.0800	0.1200	3.5700	−0.1141	−0.7393	−1.1852	−2.5583	0.0844	0.1352	0.2919	0.8762	1.8913	3.0321
1940	0.0	0.0	0.0200	2.0000	−0.1241	−0.8193	−1.2852	−4.1283	0.1017	0.1595	0.5123	1.0530	3.3823	5.3057
1941	0.0400	0.5600	1.4100	8.3100	−0.0841	−0.2593	0.1048	2.1817	0.0218	−0.0088	−0.1835	−0.0272	−0.5657	0.2286
1942	0.0500	0.5500	2.3100	8.3900	−0.0741	−0.2693	1.0048	2.2617	0.0200	−0.0745	−0.1676	−0.2706	−0.6091	2.2726
1943	0.0800	0.5700	1.5800	5.9500	−0.0441	−0.2493	0.2748	−0.1783	0.0110	−0.0121	0.0079	−0.0685	0.0445	−0.0490
1944	0.1100	1.0500	1.7400	8.1400	−0.0141	0.2307	0.4348	2.0117	−0.0033	−0.0061	−0.0284	0.1003	0.4641	0.8747
1945	0.0900	0.6600	0.6700	3.8200	−0.0341	−0.1593	−0.6352	−2.3083	0.0054	0.0217	0.0787	0.1012	0.3677	1.4662
1946	0.0200	0.3100	0.8300	5.3400	−0.1041	−0.5093	−0.4752	−0.7883	0.0530	0.0495	0.0821	0.2420	0.4015	0.3746
1947	0.0400	0.3100	0.7500	5.4600	−0.0841	−0.5093	−0.5552	−0.6683	0.0428	0.0467	0.0562	0.2828	0.3404	0.3710
1948	0.0200	0.1700	0.3300	4.3800	−0.1041	−0.6493	−0.9752	−1.7483	0.0676	0.1015	0.1820	0.6332	1.1352	1.7049
1949	0.1100	0.8600	1.6000	7.2100	−0.0141	0.0407	0.2948	1.0817	−0.0006	−0.0042	−0.0153	0.0120	0.0440	0.3189
1950	0.2100	1.3300	1.3700	5.6900	0.0859	0.5107	0.0648	−0.4383	0.0439	0.0056	−0.0376	0.0331	−0.2238	−0.0284
1951	0.3300	1.8300	3.0400	10.2700	0.2059	1.0107	1.7348	4.1417	0.2081	0.3572	0.8528	1.7534	4.1860	7.1850
1952	0.3000	1.1700	1.5900	5.7600	0.1759	0.3507	0.2848	−0.3683	0.0617	0.0501	−0.0648	0.0999	−0.1292	−0.1049
1953	0.1900	0.8400	0.8500	3.2800	0.0659	0.0207	−0.4552	−2.8483	0.0014	−0.0300	−0.1877	−0.0094	−0.0590	1.2965
1954	0.2800	1.0700	1.5500	6.3500	0.1559	0.2507	0.2448	0.2217	0.0391	0.0382	0.0346	0.0614	0.0556	0.0543
1955	0.0500	0.4300	0.9000	5.1800	−0.0741	−0.3893	−0.4052	−0.9483	0.0288	0.0300	0.0703	0.1577	0.3692	0.3843
1956	0.0300	0.2300	0.3900	3.6100	−0.0941	−0.5893	−0.9152	−2.5183	0.0555	0.0861	0.2370	0.5393	1.4840	2.3047
1957	0.4100	3.2700	5.2200	11.7700	0.2859	2.4507	3.9148	5.6417	0.7007	1.1192	1.6130	9.5940	13.8261	22.0861
1958	0.0300	0.3300	0.3800	4.8000	−0.0941	−0.4893	−0.9252	−1.3283	0.0460	0.0871	0.1250	0.4527	0.6499	1.2289
1959	0.2400	1.2500	1.2600	6.4900	0.1159	0.4307	−0.0452	0.3617	0.0499	−0.0052	0.0419	−0.0195	0.1558	−0.0163
1960	0.2300	1.0300	1.7300	5.7000	0.1059	0.2107	0.4248	−0.4283	0.0223	0.0450	−0.0454	0.0895	−0.0902	−0.1819
1961	0.1000	0.9200	0.8600	7.0900	−0.0241	0.1007	−0.4452	0.9617	−0.0024	0.0107	−0.0232	−0.0448	0.0968	−0.4281
1962	0.0700	0.7000	0.8100	5.1000	−0.0541	−0.1193	−0.4952	−1.0283	0.0065	0.0268	0.0556	0.0591	0.1227	0.5092
1963	0.0400	0.6100	1.0800	8.9300	−0.0841	−0.2093	−0.2252	2.8017	0.0176	0.0189	−0.2356	0.0471	−0.5864	−0.6309
1964	0.0500	0.4200	0.9300	5.7600	−0.0741	−0.3993	−0.3752	−0.3683	0.0296	0.0278	0.0273	0.1498	0.1471	0.1382
1965	0.4200	2.7200	3.3300	9.3800	0.2959	1.9007	2.0248	3.2517	0.5624	0.5991	0.9622	3.8485	6.1805	6.5840
1966	0.0100	0.1300	0.2400	3.8600	−0.1141	−0.6893	−1.0652	−2.2683	0.0786	0.1215	0.2588	0.7342	1.5635	2.4162
1967	0.0400	0.3600	0.9600	6.1300	−0.0841	−0.4593	−0.3452	0.0017	0.0386	0.0290	−0.0001	0.1586	−0.0008	−0.0006
Sum	3.6000	23.7600	37.8500	177.7200					2.3921	3.0255	4.5004	20.6390	34.6440	58.6966
Mean	0.1241	0.8193	1.3052	6.1283										
				Squared Sum	0.4359	15.4095	33.0335	140.6424						

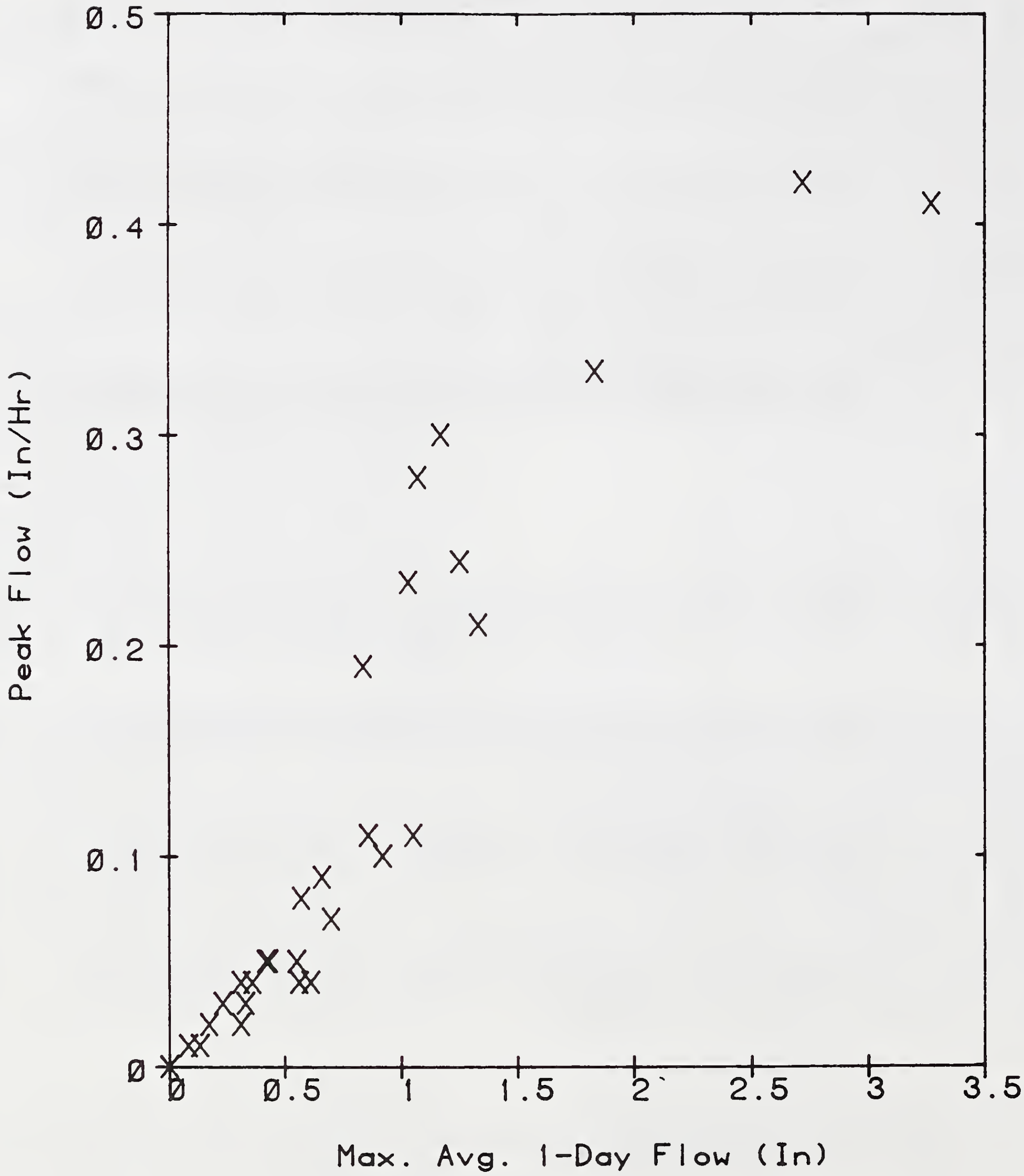

Figure 18-10.—Variable plot for example 18-4.

In the equation, peak flow varies directly with the maximum average 1-day flow and inversely with maximum monthly rainfall. The inverse relationship between Y and X_3 is not rational and should be included only if the increased significance is meaningful.

4. Analyze the residuals. Compute the standard deviation of the criterion variable (square root of equation 2), standard error of estimate (eq. 24), and coefficient of determination (eq. 25). Table 18-13 contains the data needed for this step:

$$d_f = 29 - 2 - 1 = 26$$

$$S_y = \left[\frac{\sum (Y_i - \overline{Y})^2}{N - 1}\right]^{0.5} = [0.4343/28]^{0.5} = 0.1245$$

$$S_e = \left[\frac{\sum (\hat{Y}_i - Y_i)^2}{d_f}\right]^{0.5} = [0.0508/26]^{0.5} = 0.044$$

$$r^2 = \frac{\sum (\hat{Y}_i - \bar{Y}_i)^2}{\sum (Y_i - \bar{Y})^2} = 0.3822/0.4343 = 0.880$$

The regression equation is a good predictor of the peak flow. The equation explains 88 percent of the variation in Y, and the standard error of estimate is much smaller than the standard deviation of the criterion variable, S_y.

Maximum monthly rainfall is not really needed in the equation but is included to illustrate a multiple predictor model. The correlation coefficient between peak flow and maximum 1-day flow, from the correlation matrix, indicates that the maximum 1-day flow will explain 85 percent of the variation in peak flow, i.e., $r^2 = (0.9230)^2 = 0.85$.

The sum of residuals from table 18-13 is −0.0020. The number of significant digits was not sufficient to produce truly accurate regression coefficients. More significant digits would improve the accuracy of the coefficients.

5. Plot the residuals as shown in figure 18-11. Similar plots can be made for the predictor variables and residuals. The greatest amount of underprediction (negative residual) occurs near a peak flow of 0.3 cfs. Two data points (1952 and 1954) in the region account for 46 percent of the sum of residuals squared. The greatest amount of overprediction (positive residuals) occurs at the maximum peak flow value. Large residual values (positive or negative) may be a problem when the regression equation is used in the upper range of peak flow values.

Table 18-13.—Residual data for example 18-4 (analysis of residuals for $\hat{Y} = 0.0569 + 0.1867X_1 - 0.0140X_3$)

Water year	Y = Peak flow (in/hr)	X_1 = Max. avg. 1-day flow (in)	X_3 = Max. month. rainfall (in)	$\hat{Y}$	$(\hat{Y} - Y)$	$(\hat{Y} - Y)^2$	$(\hat{Y} - \bar{Y})^2$	$(Y - \bar{Y})^2$
1939	0.0100	0.0800	3.5700	0.0219	0.0119	0.0001	0.0104	0.0130
1940	0.0	0.0	2.0000	0.0289	0.0289	0.0008	0.0090	0.0154
1941	0.0400	0.5600	8.3100	0.0451	0.0051	0.0	0.0062	0.0070
1942	0.0500	0.5500	8.3900	0.0421	−0.0079	0.0	0.0067	0.0054
1943	0.0800	0.5700	5.9500	0.0800	−0.0000	0.0	0.0019	0.0019
1944	0.1100	1.0500	8.1400	0.1390	0.0290	0.0008	0.0002	0.0001
1945	0.0900	0.6600	3.8200	0.1266	0.0366	0.0013	0.0	0.0011
1946	0.0200	0.3100	5.3400	0.0400	0.0200	0.0003	0.0070	0.0108
1947	0.0400	0.3100	5.4600	0.0383	−0.0017	0.0	0.0073	0.0070
1948	0.0200	0.1700	4.3800	0.0273	0.0073	0.0	0.0093	0.0108
1949	0.1100	0.8600	7.2100	0.1165	0.0065	0.0	0.0	0.0001
1950	0.2100	1.3300	5.6900	0.2256	0.0156	0.0002	0.0103	0.0073
1951	0.3300	1.8300	10.2700	0.2548	−0.0752	0.0056	0.0170	0.0423
1952	0.3000	1.1700	5.7600	0.1947	−0.1053	0.0110	0.0049	0.0309
1953	0.1900	0.8400	3.2800	0.1678	−0.0222	0.0004	0.0019	0.0043
1954	0.2800	1.0700	6.3500	0.1678	−0.1122	0.0125	0.0019	0.0243
1955	0.0500	0.4300	5.1800	0.0647	0.0147	0.0002	0.0035	0.0054
1956	0.0300	0.2300	3.6100	0.0493	0.0193	0.0003	0.0055	0.0088
1957	0.4100	3.2700	11.7700	0.5026	0.0926	0.0085	0.1432	0.0817
1958	0.0300	0.3300	4.8000	0.0513	0.0213	0.0004	0.0052	0.0088
1959	0.2400	1.2500	6.4900	0.1994	−0.0406	0.0016	0.0056	0.0134
1960	0.2300	1.0300	5.7000	0.1694	−0.0606	0.0036	0.0020	0.0112
1961	0.1000	0.9200	7.0900	0.1294	0.0294	0.0008	0.0	0.0005
1962	0.0700	0.7000	5.1000	0.1162	0.0462	0.0021	0.0	0.0029
1963	0.0400	0.6100	8.9300	0.0458	0.0058	0.0	0.0061	0.0070
1964	0.0500	0.4200	5.7600	0.0547	0.0047	0.0	0.0048	0.0054
1965	0.4200	2.7200	9.3800	0.4334	0.0134	0.0001	0.0956	0.0875
1966	0.0100	0.1300	3.8600	0.0271	0.0171	0.0002	0.0094	0.0130
1967	0.0400	0.3600	6.1300	0.0383	−0.0017	0.0	0.0073	0.0070
Sum					−0.0020	0.0508	0.3822	0.4343

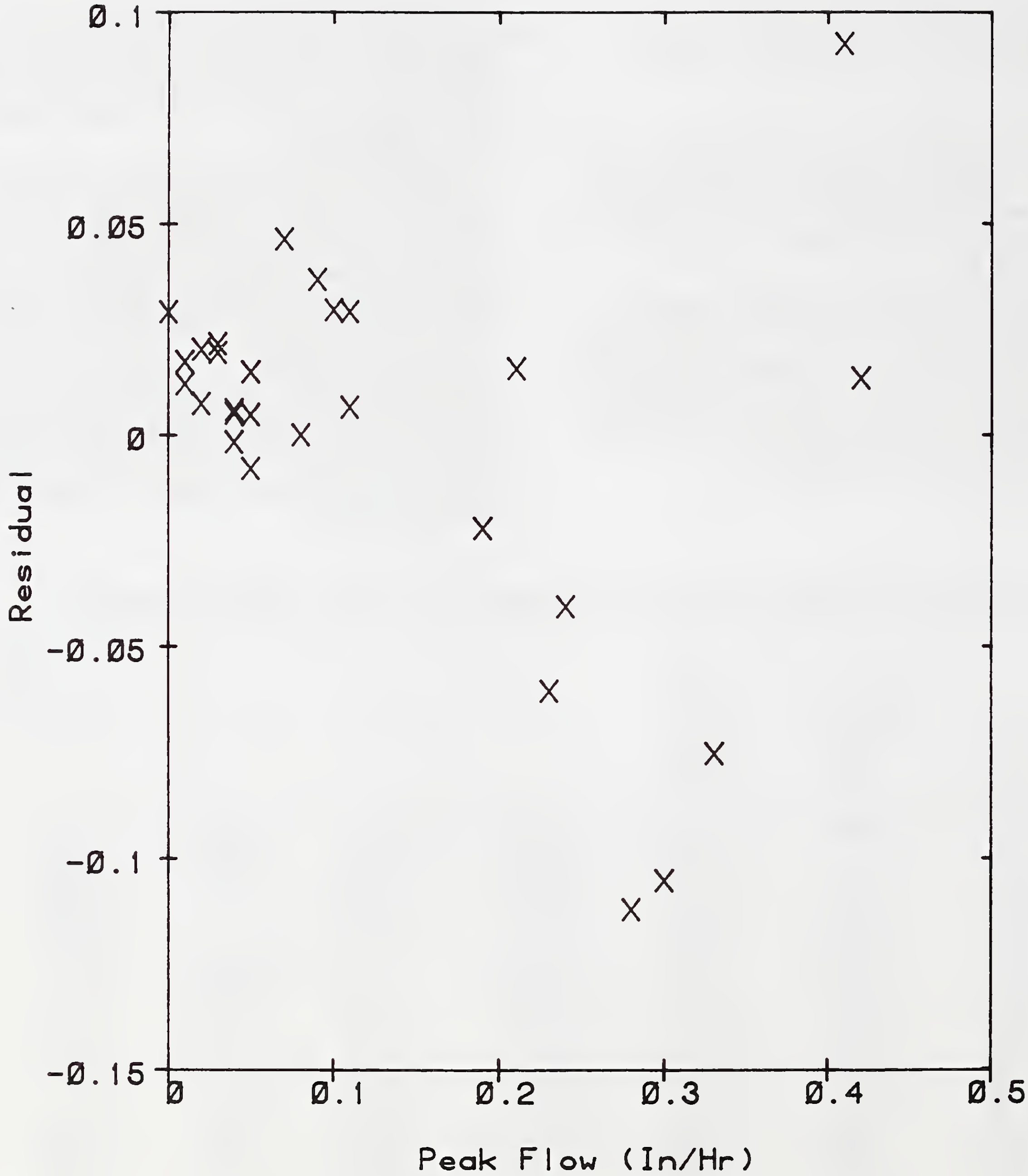

Figure 18-11.—Residual plot for example 18-4.

Analysis Based on Regionalization

Purpose

Many watersheds analyzed by SCS are in locations for which few data are available, so techniques have been developed to transfer or regionalize available data to other locations.

One purpose of regionalization is to synthesize a frequency curve at an ungaged location or at a location where data are inadequate for developing a frequency curve by using the methods in the Frequency Analysis section. The most common forms of regionalization use watershed and hydrometeorological characteristics as predictor variables. Data may be regionalized by either direct or indirect estimation.

Direct Estimation

The most commonly used technique is to relate selected values at various exceedance frequencies to the physical characteristics of the watershed. For example, the 10-year, 7-day mean flow may be related to drainage area and percentage of forest cover. The predictor variables can include both physical and hydrometeorological characteristics.

Previous studies have included the following as predictors: drainage area, mean watershed slope, mean basin elevation, length and slope of the main watercourse, the weighted runoff curve number, percentage of watershed in lakes or various cover types, and geological characteristics. Meteorological characteristics include: mean annual precipitation, mean annual snowfall, mean annual temperature, mean monthly temperature, mean monthly precipitation, and the 24-hour duration precipitation for various frequencies. Latitude, longitude, and watershed orientation have been included as location parameters. This list of various predictor variables is not complete but has been included to give some concept of the characteristics that can be used.

Example 18-5.—Development of a direct probability estimate by use of stepwise regression.

A sample power form prediction equation is:

$$\hat{Y} = b_0 X_1^{b_1} X_2^{b_2} X_3^{b_3} \ldots X_n^{b_n}$$

where $\hat{Y}$ is the estimated criterion variable; X_1, X_2, $X_3 \ldots X_n$ are the predictor variables; and b_0, b_1, $b_2 \ldots b_n$ are the regression coefficients.

The regression coefficients are developed from a multiple linear regression of the logarithms of the data. When the variables are transformed back to original units, the regression coefficients become powers.

Table 18-14 contains 9 variables for 18 North Coastal California Watersheds used to develop a power equation for estimating the 1-percent maximum 7-day mean runoff ($V_{0.01}$). A locally available stepwise regression computer program (Dixon 1975) is used in the analysis.

The correlation matrix of the logarithms of the data is in table 18-15. The highest correlations of logarithms between runoff volume and the other variables are between channel length (−0.62) and drainage area (−0.53). These two variables are highly correlated (0.96) themselves, so only one would be expected to be used in the final equation. Rainfall intensity (0.48) and annual precipitation (0.45) are the variables with the next highest correlations to $V_{0.01}$. One or both of these variables may appear in the final regression equation.

The results of the stepwise regression analysis are in tables 18-16 and 18-17. Table 18-16 contains the regression coefficients for each step of the regression. Table 18-17 contains the regression equation data for each step. Equation 5 was selected as the best because the regression coefficents are rational, and including additional variables does not significantly decrease the standard error of estimate. All equations are significant based on the total F test at the 1-percent level. The least significant variable is slope (S) based on a 1-percent level F with 4 and 13 degrees of freedom. From a standard F table, for these degrees of freedom, $F_{0.01} = 3.18$. The partial F value required to enter the "slope" variable is 5.3. Equation 5 explains 83.6 percent of the variation (r^2) in the logarithm of $V_{0.01}$, and addition of all remaining variables will only raise this to 87.3 percent.

Examine the residuals to evaluate the quality of the selected regression equation. Table 18-18 contains the predicted and observed $V_{0.01}$ logarithms as well as the residuals and their sum. A plot of the residuals with the predicted values in figure 18-12 shows no correlation between $V_{0.01}$ logarithms and the residuals. The residual variation is also constant over the range of the $V_{0.01}$ logarithms.

The final power equation is:

$$V_{0.01} = 10^{(0.6752)} L^{(-0.4650)} P^{(0.6735)} F^{(0.1432)} S^{(-0.1608)}$$

For data from station 11372000 (table 18-14), the

estimated $V_{0.01}$ is:

$$V_{0.01} = (4.7337)\,(48.7)^{(-0.4650)}\,(56)^{(0.6735)}\,(99)^{(0.1432)}\,(63)^{(-0.1608)}$$

$$V_{0.01} = 11.60 \text{ watershed inches.}$$

Similar procedures can be used to develop regression equations for 0.50, 0.20, 0.10, 0.04, and 0.02 exceedance probabilities. Because each equation may not contain the same predictor variables, inconsistencies may develop from one exceedance probability to another. A method of eliminating in-

Table 18-14.—Basic data for example 18-5

Station no.	Drainage area (A)	Mean annual precipitation (P)	Two-year, 24-hr rainfall intensity (I)	Evaporation (E)	Channel slope (S)	Channel length (L)	Altitude (Al)	Percent forest (F)	Runoff volume ($V_{0.01}$)
	mi^2		*inches*		*ft/mi*	*mi*	*1,000 ft*	*% + 1*	*inches*
11372000	228.0	56	3.5	48	63	48.7	2.1	99	11.1966
11374400	249.0	41	2.8	48	58	43.5	1.6	53	7.6804
11379500	92.9	36	2.8	51	170	19.6	2.0	92	10.3144
11380500	126.0	28	2.7	51	93	42.7	1.8	84	6.6278
11382000	194.0	35	2.8	49	126	36.5	2.7	98	11.5990
11448500	6.36	41	4.5	46	374	4.2	2.1	95	18.9540
11448900	11.9	37	4.0	45	125	5.3	1.9	85	20.8693
11451500	197.0	39	3.0	52	40	34.0	1.7	96	10.1729
11451720	100.0	30	3.8	51	17	38.0	1.3	90	8.8838
11453500	113.0	52	3.5	49	55	21.6	1.4	89	18.8469
11453600	78.3	35	4.0	49	30	18.0	0.8	60	17.7086
11456000	81.4	48	3.3	49	46	19.4	0.5	79	16.2089
11456500	52.1	35	3.3	49	140	14.3	1.0	87	11.1178
11457000	17.4	35	3.3	49	72	10.8	1.2	29	13.1009
11458200	9.79	30	2.4	45	258	8.9	1.1	98	14.6669
11458500	58.4	35	3.0	46	82	17.3	0.3	72	15.9474
11459000	30.9	28	3.0	43	95	10.3	0.4	1	7.3099
11460000	18.1	42	3.0	42	125	7.5	0.5	50	19.0027

Table 18-15.—Correlation matrix of logarithms for example 18-5

Variable	Drainage area (A)	Mean annual precipitation (P)	Two-year, 24-hr rainfall intensity (I)	Evaporation (E)	Channel slope (S)	Channel length (L)	Altitude (Al)	Percent forest (F)	Runoff volume ($V_{0.01}$)
	mi^2		*inches*		*ft/mi*	*mi*	*1,000 ft.*	*% + 1*	*inches*
Area	1.00								
Precipitation	0.23	1.00							
Intensity	−0.25	0.32	1.00						
Evaporation	0.63	0.01	−0.03	1.00					
Slope	−0.60	−0.10	−0.19	−0.44	1.00				
Length	0.96	0.11	−0.32	0.68	−0.61	1.00			
Altitude	0.22	0.14	0.11	0.50	0.16	0.27	1.00		
Forest	0.19	0.36	0.11	0.49	0.01	0.22	0.49	1.00	
Runoff volume	−0.53	0.45	0.48	−0.37	0.22	−0.62	−0.17	0.34	1.00

constencies is to smooth estimated values over the range of exceedance probabilities. Figure 18-13 illustrates the smoothing for station 11372000.

Table 18-16.—Stepwise regression coefficients for example 18-5

Equation no.	Constant	L	P	F	S	Al	A	E	I
1	1.0997								
2	1.4745	−0.3010							
3	−0.0022	−0.3281	0.9615						
4	0.1739	−0.3605	0.7380	0.1210					
5	0.6752	−0.4650	0.6735	0.1432	−0.1608				
6	0.5178	−0.4257	0.6731	0.1675	−0.1231	−0.1046			
7	0.6604	−0.5722	0.5803	0.1756	−0.1242	−0.1012	0.0985		
8	2.6010	−0.5796	0.4824	0.1980	−0.1509	−0.0681	0.1233	−1.0785	
9	2.6392	−0.5971	0.4949	0.1983	−0.1623	−0.0608	0.1257	−1.0705	−0.0637

Table 18-17.—Regression equation evaluation data for example 18-5

Equation no.	Predictor variables	r^2	Δr^2	S_e	SS/df Reg	SS/df Res	F_t Ratio	F_p Ratio
1	---			0.1566*				
2	L	0.390	0.390	0.1260	0.1627/1	0.2542/16	10.2	10.2
3	L,P	0.661	0.271	0.0971	0.2754/2	0.1415/15	14.6	11.9
4	L,P,F	0.769	0.108	0.0830	0.3204/3	0.0964/14	15.5	6.5
5	L,P,F,S	0.836	0.067	0.0725	0.3485/4	0.0684/13	16.6	5.3
6	L,P,F,S,Al	0.858	0.022	0.0703	0.3575/5	0.0593/12	14.5	1.8
7	L,P,F,S,Al,A	0.864	0.006	0.0718	0.3601/6	0.0567/11	11.6	0.5
8	L,P,F,S,Al,A,E	0.873	0.009	0.0728	0.3639/7	0.0530/10	9.8	0.7
9	L,P,F,S,Al,A,E,I	0.873	0.000	0.0766	0.3640/8	0.0529/9	7.7	0.2

r^2 - Coefficient of determination
Δr^2 - Change in r^2
S_e - Standard error of estimate
SS/df - Sum of squares to degrees of freedom ratio for regression (Reg) or residuals (Res)
F_t - Total F test value
F_p - Partial F test value
*S_y of criterion variable, $V_{0.01}$

Table 18-18.—Residuals for example 18-5

Staion no.	Predicted runoff volume (logs)	Observed runoff volume (logs)	Residual
11372000	1.0646	1.0491	−0.0155
11374400	0.9631	0.8854	−0.0777
11379500	1.0453	1.0137	−0.0316
11380500	0.8510	0.8214	−0.0296
11382000	0.9363	1.0644	0.1281
11448500	1.3413	1.2777	−0.0636
11448900	1.3339	1.3195	−0.0144
11451500	1.0611	1.0074	−0.0537
11451720	1.0177	0.9486	−0.0691
11453500	1.2099	1.2752	0.0653
11453600	1.1487	1.2482	0.0995
11456000	1.2133	1.2098	−0.0035
11456500	1.1108	1.0460	−0.0648
11457000	1.1455	1.1173	−0.0282
11458200	1.1261	1.1663	0.0402
11458500	1.0979	1.2027	0.1048
11459000	0.8610	0.8639	0.0029
11460000	1.2679	1.2788	0.0109
		Sum	0.0000

The U.S. Geological Survey uses stepwise multiple regression to develop predictive equations for selected flow values. The results are published in open file reports that generally include predictive equations for major river basins, physiographic regions, or states. Meteorological and physical characteristics listed in the reports can be used to develop applicable predictive equations for SCS hydrologic studies.

Indirect Estimation

The second technique is to use regression equations to relate the statistical characteristics of selected values to various basin characteristics. The probability level estimates are then derived from the frequency curve, based on the predicted statistical characteristics.

Example 18-6.—Development of indirect probability estimates.

In the north coastal region of California, the mean and standard deviation of the 1-day and 15-day high flow frequency curves were related to basin characteristics. Twenty-five stations were used in the relationships shown in figures 18-14 through 18-17. The relationships of drainage area, mean annual precipitation, 1-day and 15-day high flow means and standard deviations were developed by regression. The predictor variables were selected because of availability of data. Tests were performed on each regression equation to verify that the mean of residuals is zero, the residuals are independent of each variable, the variance is constant, and that S_e is smaller than S_y, the standard deviation of the criterion.

Develop 1- and 15-day high flow frequency curves for a 50-square-mile drainage area in the north coastal region of California with a mean annual precipitation of 60 inches.

$S_1 = 1{,}400$ cfs from fig. 18-15

$\gamma_1 = (\overline{X})^2/S^2$ solution of eq. 13 for γ

$$\gamma_1 = \frac{(3{,}100)^2}{(1{,}400)^2} = 4.9$$

$G_1 = 2/\sqrt{4.9} = 0.90$ from eq. 14

$\overline{X}_{15} = 900$-cfs days from fig. 18-16

$S_{15} = 340$ cfs from fig. 18-17

$$\gamma_{15} = \frac{(900)^2}{(340)^2} = 7.0$$

$G_{15} = 2/\sqrt{7.0}$
use 0.8

Using equation 15 as shown in table 18-19, determine the 1-day and 15-day high flow values for selected exceedance frequencies.

where: $\overline{X}_1 = 3{,}100$ $\overline{X}_{15} = 900$
$S_1 = 1{,}400$ $S_{15} = 340$

Discussion

The basic uses of regionalization are to transfer data either from gaged watersheds to ungaged watersheds or to locations within gaged watersheds, and to calibrate water resource models. But in using regionalization, one must understand certain basic limitations.

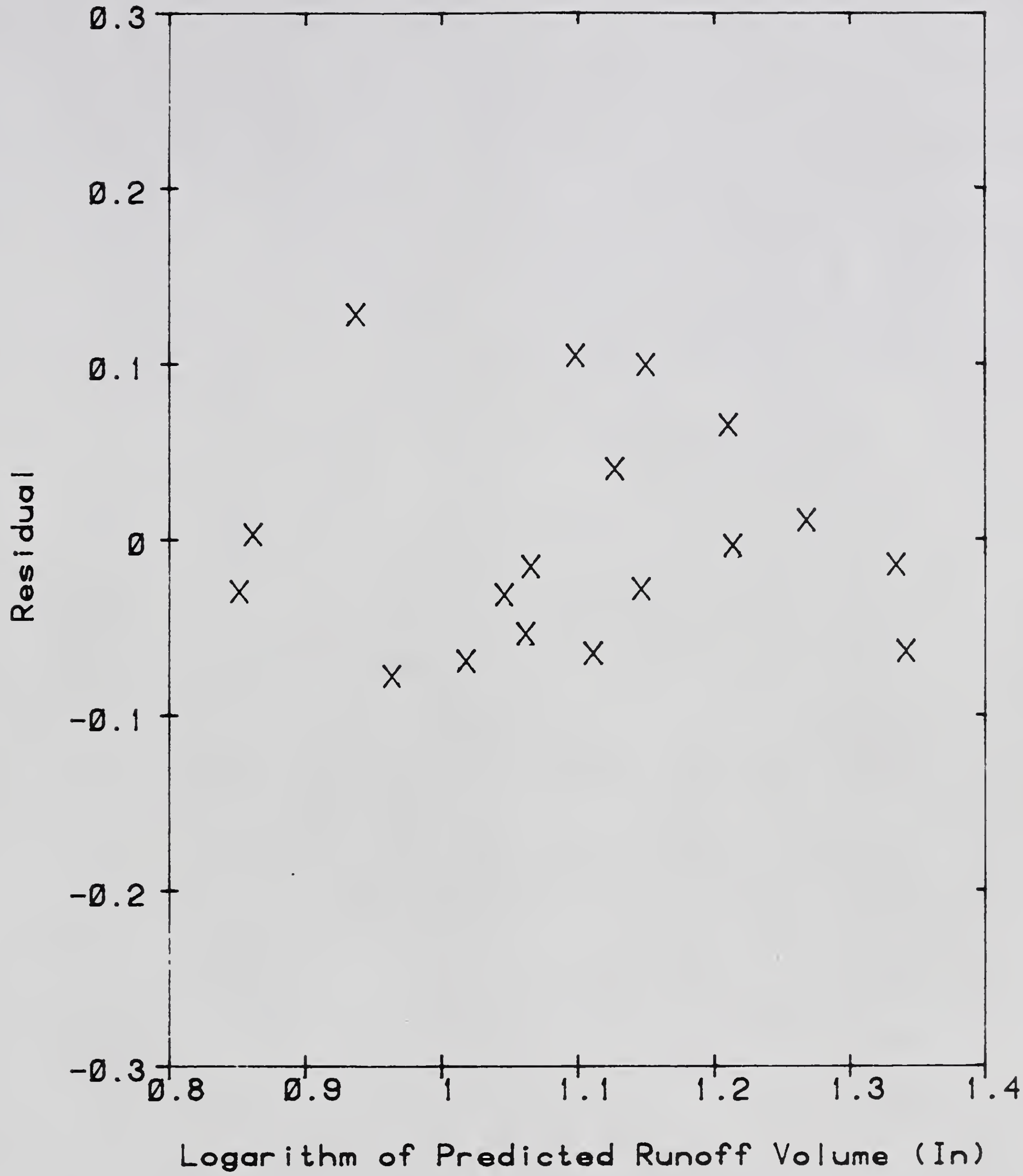

Figure 18-12.—Residual plot for example 18-5.

The prediction equation generally should be used only within the range of the predictor variables used to develop the equation. The prediction equation represents the "average" condition for the data. If the ungaged watershed varies significantly from the average condition, then the variation must be explained by one or more of the variables in the prediction equation. If the variation is not explained, the equation should not be used.

When the prediction equation is used to calibrate a watershed model, values estimated by the regression equation should deviate from the values computed by the model. The magnitude of this deviation is a function of how much the ungaged watershed differs from the "average" condition. For example, if most of the watersheds used to develop the prediction equation are flat and long and the ungaged watershed is steep and short, the peak

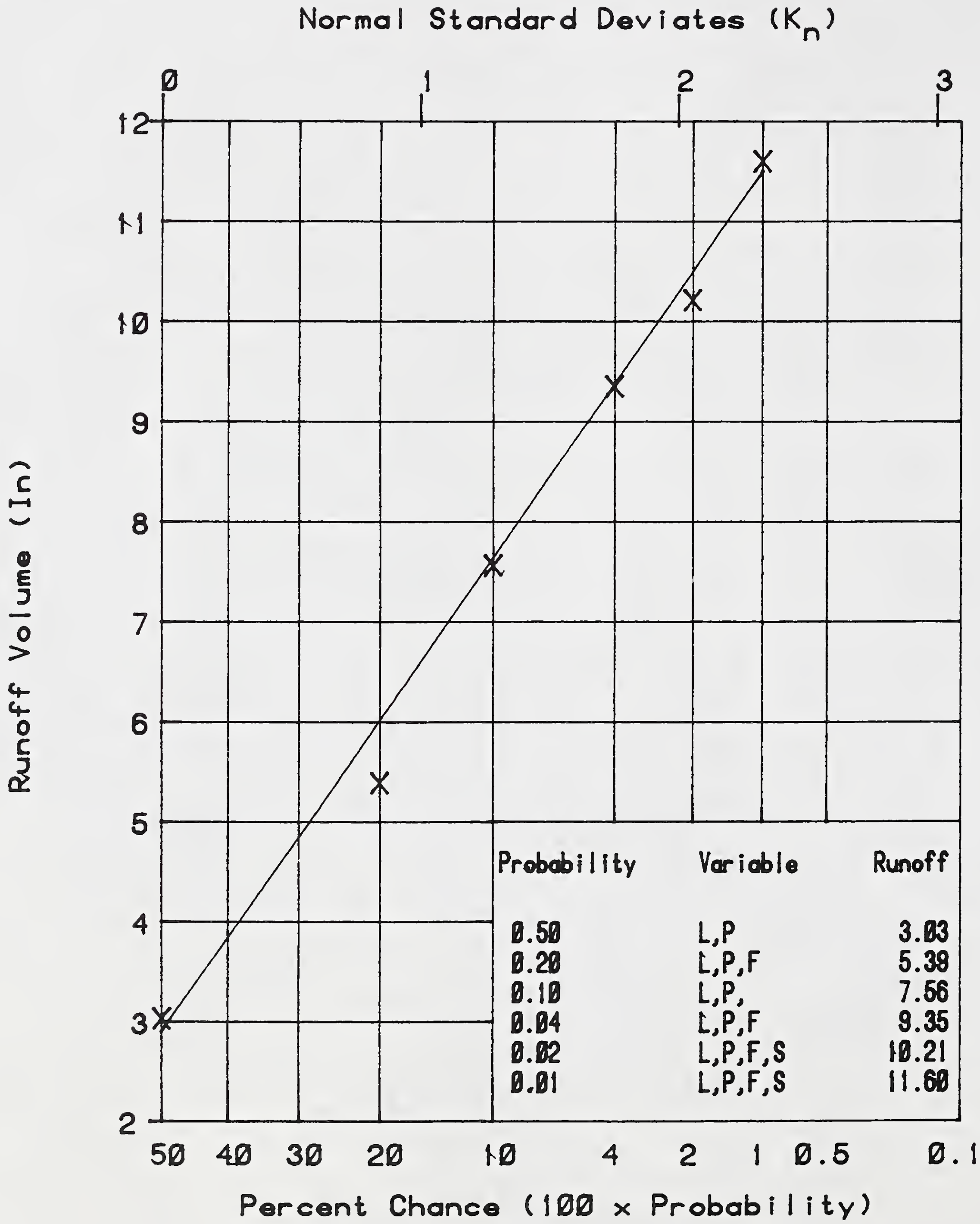

Figure 18-13.—Estimate smoothing for example 18-5.

flow computed with the watershed model could differ significantly from that estimated by the prediction equation. The prediction equation should not be used when the watershed characteristics are outside the range of those used to develop the equation.

The coefficients of the prediction equation must be rational. For example, peak flow is inversely proportional to the length of the main watercourse, if all other variables are constant. This means that when a logarithmic transformation is used, the power of the length variable should be negative. If

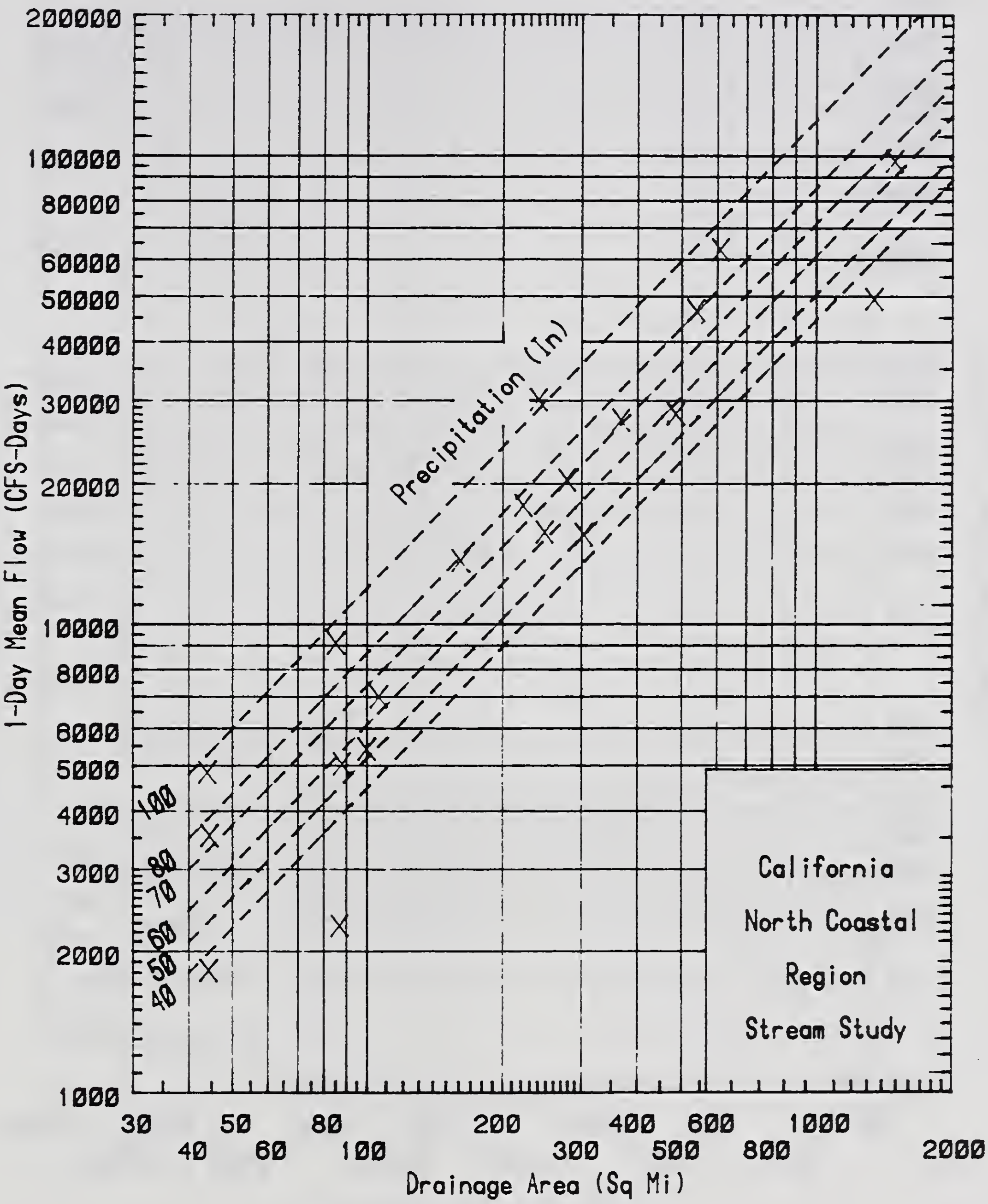

Figure 18-14.—Drainage area and mean annual precipitation for 1-day mean flow for example 18-6.

a predictor variable has an irrational relationship in the equation, the correlation coefficients of all the predictor variables should be examined before the equation is used. A high correlation coefficient between two predictor variables means that one of the variables can be used to explain how the criterion variable varies with both predictor variables. The accuracy of the prediction equation is not improved by adding the second predictor variable; the equation merely becomes more complicated.

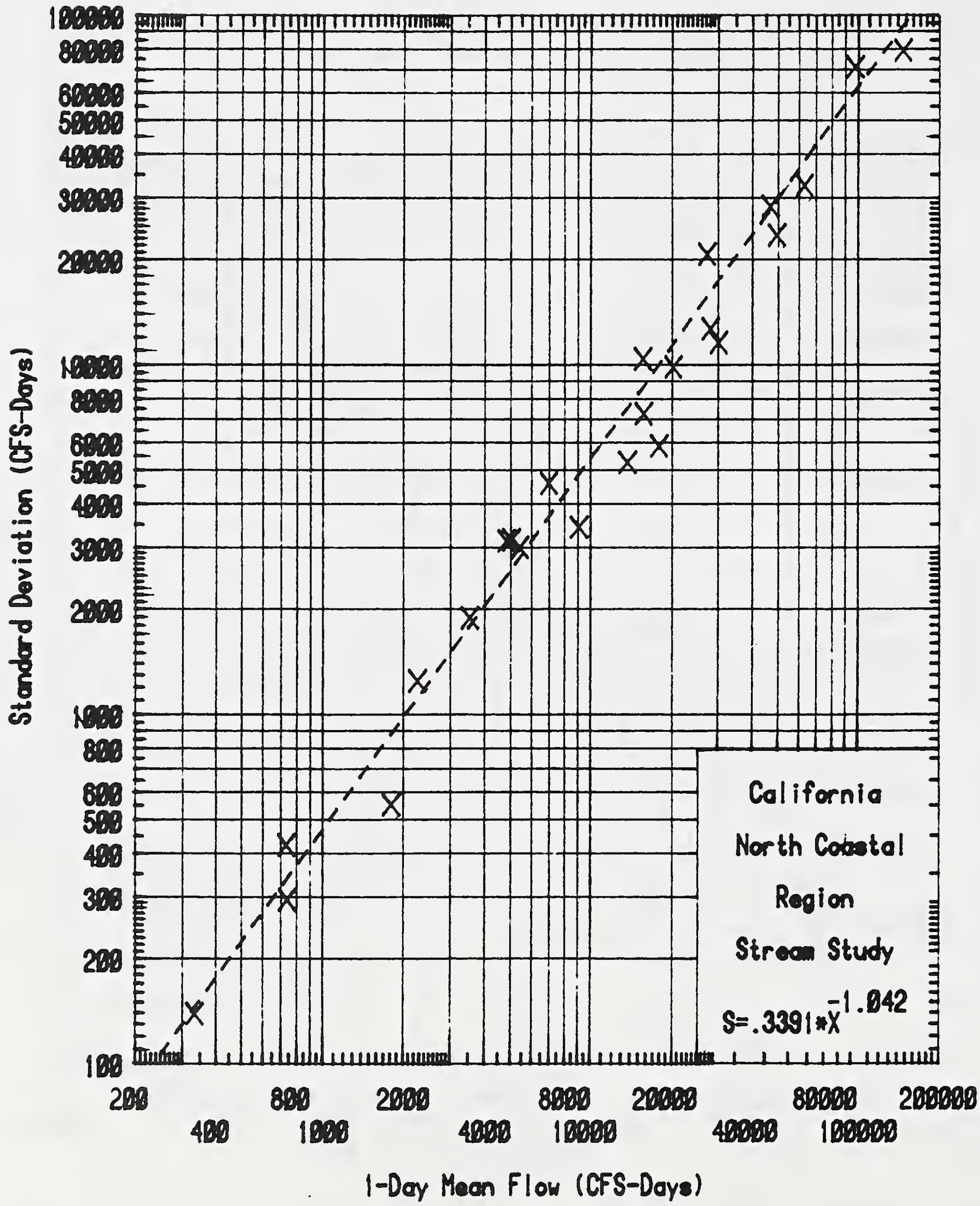

Figure 18-15.—One-day mean flow and standard deviation for example 18-6.

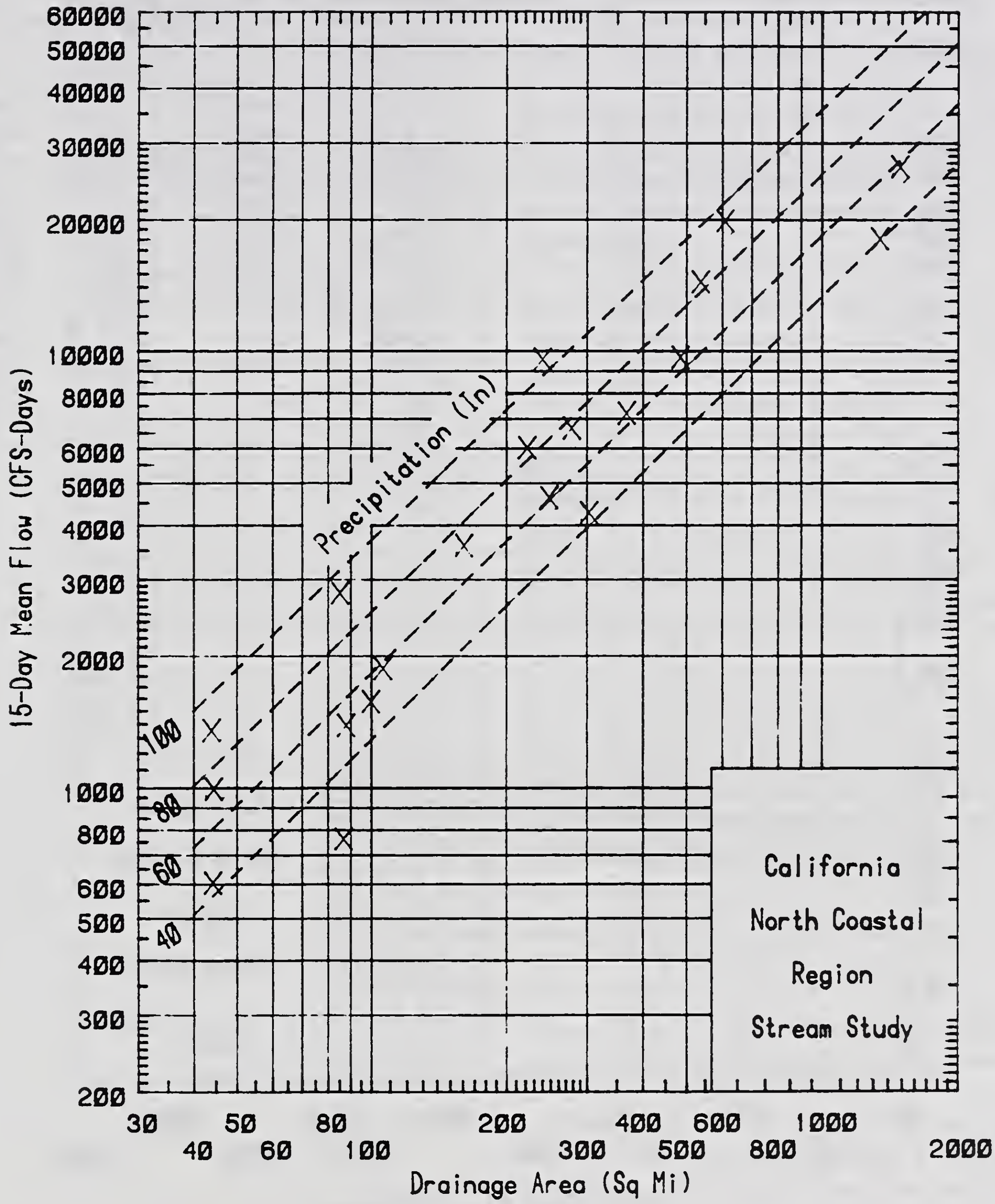

Figure 18-16.—Drainage area and mean annual precipitation for 15-day mean flow for example 18-6.

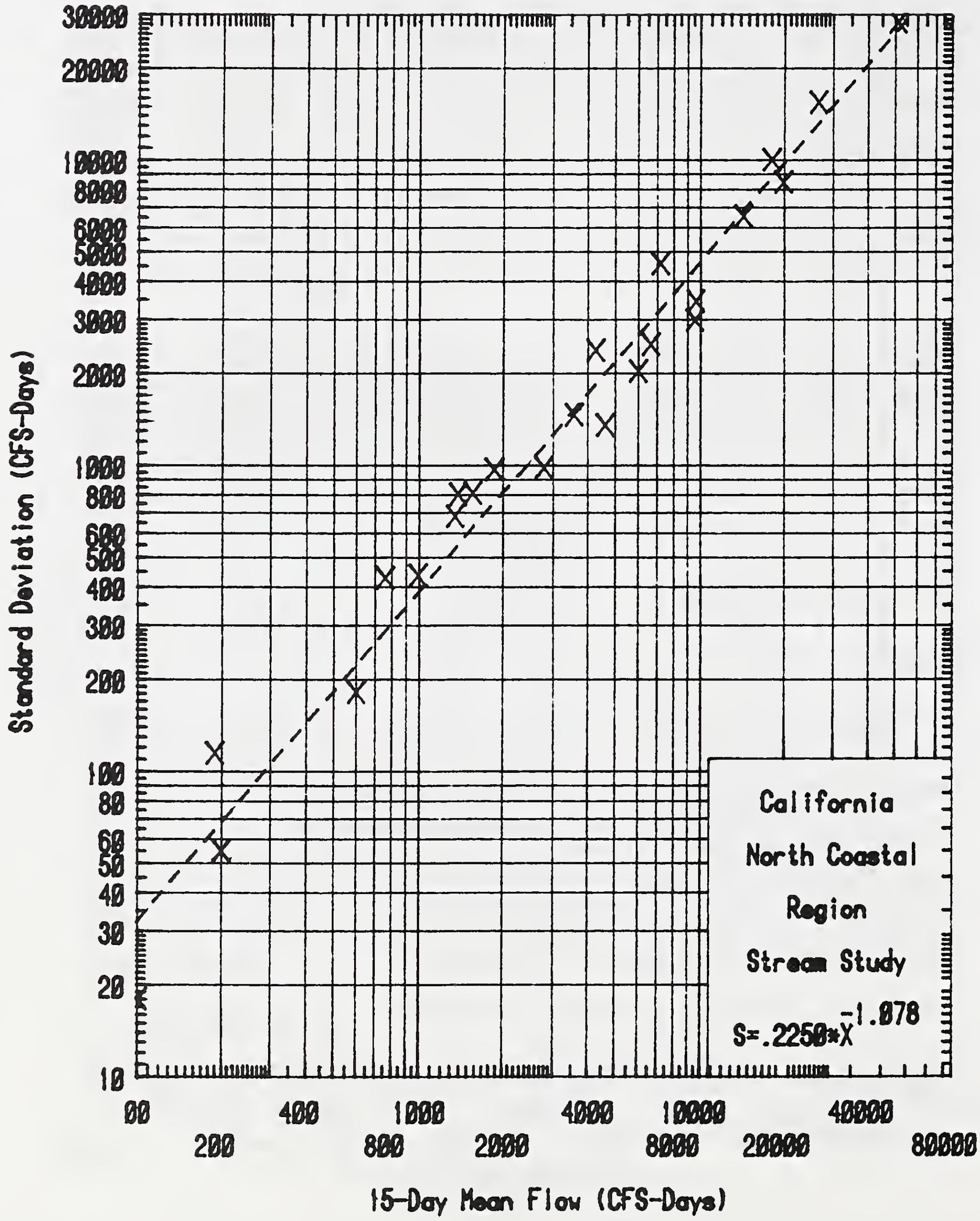

Figure 18-17.—Fifteen-day mean flow and standard deviation for example 18-6.

Table 18-19.—Frequency curve solutions for example 18-6

Exceed prob. (q)	TR-38 K value (G=0.9)	$V_1 = \overline{X}_1 + KS_1$	TR-38 K value (G=0.8)	$V_{15} = \overline{X}_{15} + KS_{15}$
99	−1.66001	776	−1.73271	311
95	−1.35299	1206	−1.38855	428
80	−0.85426	1904	−0.85607	609
50	−0.14807	2893	−0.13199	855
20	0.76902	4177	0.77986	1165
10	1.33889	4974	1.33640	1354
4	2.01848	5926	1.99311	1578
2	2.49811	6597	2.45298	1734
1	2.95735	7240	2.89101	1883

Risk

Flood frequency analysis identifies the population from a sample of data. The population cannot be identified exactly when only a sample is available, and this represents one important element of uncertainty. A second source of uncertainty exists also; even if the population were known exactly, there is a finite chance that an event of a certain size will be exceeded.

The measurement of such uncertainty is called *risk*. Typical questions include: (1) A channel is designed with a capacity of a 0.02 exceedance probability. Is it unreasonable to expect its capacity will be exceeded once or more in the next 10 years? (2) What is the risk that an emergency spillway designed to pass a 2-percent chance flow will experience this flow twice or more in the next 10 years? (3) Throughout the United States the Soil Conservation Service has built many flood-control structures. What percent will experience a 1-percent chance flood in the next 5 years? The next 10 years?

These problems can be solved by means of the binomial distribution. Basic assumptions in the use of the binomial distribution are given in the general discussion on distributions. These assumptions are usually valid for assessing risk in hydrology.
The binomial expression for risk is:

$$R_I = \frac{N!}{I!\,(N - I)!}\, q^I (1 - q)^{(N - I)} \qquad (18\text{-}29)$$

where R_I is the estimated risk of obtaining in N time periods exactly I number of events with an exceedance probability q.

Example 18-7.—Risk of future nonoccurrence.

What is the probability that a 10-percent chance flood ($q = 0.10$) will not be exceeded in the next 5 years?

From equation 29, for $N = 5$, $q = 0.10$, and $I = 0$,

$$R_0 = \frac{(5)!}{0!(5)!}\, 0.10^0 (1 - 0.10)^{(5 - 0)}$$

The probability of nonoccurrence is 0.59 or 59 percent; the probability of occurrence is $1 - R_0$ or 0.41.

Example 18-8.—Risk of multiple occurrence.

What is the probability that a 2-percent chance peak flow ($q = 0.02$) will be exceeded *twice or more* in the next 10 years?

For nonexceedance of the 2-percent chance event,

$$N = 10,\ q = 0.02,\ I = 0$$

$$R_0 = \frac{(10)!}{0!(10)!}(0.02)^0(1 - 0.02)^{10}$$
$$= 0.817$$

For only one exceedance of the 2-percent chance event,

$$N = 10,\ q = 0.02,\ I = 1$$

$$R_1 = \frac{(10)!}{(1)!(9)!}(0.02)^1(1 - 0.02)^9$$

$$= 0.167$$

For 2 or more exceedances of the 2-percent chance event,

$$R_{(2\text{ or more})} = 1 - (R_0 + R_1)$$

$$R_{(2\text{ or more})} = 1 - (0.817 + 0.167)$$
$$= 0.016$$

In other words, there is a 1.6-percent chance of experiencing two or more peaks equal to or greater than the 2-percent chance peak flow within any 10-year period. If flood events are not related, it is likely that within the next 10 years no more than 16 locations in a thousand will, on the average, experience two or more floods equal to or greater than the 2-percent chance flood.

Example 18-9.—Risk of a selected exceedance probability.

There is a 20-year record on a small creek. What is the probability that the greatest flood of record is not a 5-percent chance event ($q = 0.05$)?

For nonoccurrence of the 5-percent chance event,

$$N = 20,\ q = 0.05,\ I = 0$$

$$R_0 = \frac{20!}{0!20!}(0.05)^0(1 - 0.05)^{20}$$
$$= 0.358$$

Therefore, there is a 36-percent chance of the 5-percent chance event not occurring and, conversely, a 64-percent chance that one or more will occur.

Example 18-10.—Exceedance probability of a selected risk.

What exceedance probability has a 50-percent chance of occurrence in a 20-year period?

For 50-percent occurrence in 20 years,

$$N = 20,\ q = ?,\ I = 0,\ R_0 = 0.5$$

$$0.5 = \frac{(20)!}{0!20!}(q)^0(1 - q)^{(20 - 0)}$$

$$0.5 = (1 - q)^{20}$$

$$1 - q = (0.5)^{1/20} = 0.966$$

$$q = 0.034$$

or there is a 50-percent chance that a 3-percent chance event will occur within the 20-year period.

Metric Conversion Factors

The English system of units is used in this report. To convert to the International System of units (metric), use the following factors:

To convert English units	*To metric units*	*Multiply by*
acres (acre)	hectares (ha)	0.405
square miles (sq. mi)	square kilometers (km^2)	2.59
cubic feet per second (cfs)[1]	cubic meters per second (m^3/sec)	0.0283
cubic feet per second-days (cfs-days)	cubic meters (m^3)	2,450
inches (in)	millimeters (mm)	25.4

[1] In converting stream discharge values, which are recorded in English units with only three significant digits, be careful that you do not imply a greater precision than is present.

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Exhibit 18-1.—Five-percent two-sided critical values for outlier detection

N	K_n	Low prob.	High prob.
10	2.294	0.9891048	0.0108952
11	2.343	0.9904353	0.0095647
12	2.387	0.9915068	0.0084932
13	2.426	0.9923669	0.0076331
14	2.461	0.9930725	0.0069275
15	2.493	0.9936665	0.0063335
16	2.523	0.9941821	0.0058179
17	2.551	0.9946293	0.0053707
18	2.577	0.9950169	0.0049831
19	2.600	0.9953388	0.0046612
20	2.623	0.9956420	0.0043580
21	2.644	0.9959034	0.0040966
22	2.664	0.9961391	0.0038609
23	2.683	0.9963517	0.0036483
24	2.701	0.9965434	0.0034566
25	2.717	0.9967061	0.0032939
26	2.734	0.9968715	0.0031285
27	2.751	0.9970293	0.0029707
28	2.768	0.9971799	0.0028201
29	2.781	0.9972904	0.0027096
30	2.794	0.9973969	0.0026031
31	2.808	0.9975075	0.0024925
32	2.819	0.9975913	0.0024087
33	2.833	0.9976943	0.0023057
34	2.846	0.9977863	0.0022137
35	2.858	0.9978684	0.0021316
36	2.869	0.9979411	0.0020589
37	2.880	0.9980116	0.0019884
38	2.890	0.9980738	0.0019262
39	2.900	0.9981341	0.0018659
40	2.910	0.9981928	0.0018072
41	2.919	0.9982442	0.0017558
42	2.925	0.9982777	0.0017223
43	2.937	0.9983429	0.0016571
44	2.945	0.9983852	0.0016148
45	2.954	0.9984316	0.0015684
46	2.960	0.9984618	0.0015382
47	2.970	0.9985110	0.0014890
48	2.978	0.9985493	0.0014507
49	2.985	0.9985821	0.0014179
50	2.993	0.9986187	0.0013813
51	3.000	0.9986501	0.0013499
52	3.007	0.9986808	0.0013192
53	3.013	0.9987066	0.0012934
54	3.020	0.9987361	0.0012639
55	3.025	0.9987568	0.0012432
56	3.032	0.9987853	0.0012147
57	3.040	0.9988171	0.0011829
58	3.046	0.9988404	0.0011596
59	3.051	0.9988596	0.0011404
60	3.058	0.9988859	0.0011141
61	3.063	0.9989043	0.0010957
62	3.070	0.9989297	0.0010703
63	3.075	0.9989474	0.0010526
64	3.082	0.9989719	0.0010281
65	3.086	0.9989856	0.0010144
66	3.090	0.9989992	0.0010008
67	3.096	0.9990192	0.0009808
68	3.101	0.9990356	0.0009644
69	3.105	0.9990486	0.0009514
70	3.110	0.9990645	0.0009355
71	3.115	0.9990802	0.0009198
72	3.121	0.9990988	0.0009012
73	3.125	0.9991109	0.0008891
74	3.130	0.9991260	0.0008740
75	3.134	0.9991378	0.0008622
76	3.138	0.9991494	0.0008506
77	3.142	0.9991609	0.0008391
78	3.148	0.9991780	0.0008220
79	3.152	0.9991892	0.0008108
80	3.157	0.9992030	0.0007970
81	3.161	0.9992138	0.0007862
82	3.164	0.9992219	0.0007781
83	3.168	0.9992325	0.0007675
84	3.172	0.9992430	0.0007570
85	3.176	0.9992533	0.0007467
86	3.180	0.9992636	0.0007364
87	3.184	0.9992737	0.0007263
88	3.188	0.9992837	0.0007163
89	3.191	0.9992911	0.0007089
90	3.194	0.9992984	0.0007016
91	3.198	0.9993080	0.0006920
92	3.202	0.9993176	0.0006824
93	3.205	0.9993247	0.0006753
94	3.208	0.9993317	0.0006683
95	3.211	0.9993386	0.0006614
96	3.214	0.9993455	0.0006545
97	3.217	0.9993523	0.0006477
98	3.220	0.9993590	0.0006410
99	3.224	0.9993679	0.0006321
100	3.228	0.9993767	0.0006233

Note: K_n values are positive for high outliers and negative for low outliers.

EXHIBIT 18-2 EXPECTED VALUES OF NORMAL ORDER STATISTICS

K/N	10	11	12	13	14	15	16
1	1.53875	1.58644	1.62923	1.66799	1.70338	1.73591	1.76599
2	1.00136	1.06192	1.11573	1.16408	1.20790	1.24794	1.28474
3	0.65606	0.72884	0.79284	0.84983	0.90113	0.94769	0.99027
4	0.37576	0.46198	0.53684	0.60285	0.66176	0.71488	0.76317
5	0.12267	0.22489	0.31225	0.38833	0.45557	0.51570	0.57001
6		0.0	0.10259	0.19052	0.26730	0.33530	0.39622
7				0.0	0.08816	0.16530	0.23375
8						0.0	0.07729

K/N	17	18	19	20	21	22	23
1	1.79394	1.82003	1.84448	1.86748	1.88917	1.90969	1.92916
2	1.31878	1.35041	1.37994	1.40760	1.43362	1.45816	1.48137
3	1.02946	1.06573	1.09945	1.13095	1.16047	1.18824	1.21445
4	0.80738	0.84812	0.88586	0.92098	0.95380	0.98459	1.01356
5	0.61946	0.66479	0.70661	0.74538	0.78150	0.81527	0.84697
6	0.45133	0.50158	0.54771	0.59030	0.62982	0.66667	0.70115
7	0.29519	0.35084	0.40164	0.44833	0.49148	0.53157	0.56896
8	0.14599	0.20774	0.26374	0.31493	0.36203	0.40559	0.44609
9	0.0	0.06880	0.13072	0.18696	0.23841	0.28579	0.32965
10			0.0	0.06200	0.11836	0.16997	0.21755
11					0.0	0.05642	0.10813
12							0.0

K/N	24	25	26	27	28	29	30
1	1.94767	1.96531	1.98216	1.99827	2.01371	2.02852	2.04276
2	1.50338	1.52430	1.54423	1.56326	1.58145	1.59888	1.61560
3	1.23924	1.26275	1.28511	1.30641	1.32674	1.34619	1.36481
4	1.04091	1.06679	1.09135	1.11471	1.13697	1.15822	1.17855
5	0.87682	0.90501	0.93171	0.95705	0.98115	1.00414	1.02609
6	0.73354	0.76405	0.79289	0.82021	0.84615	0.87084	0.89439
7	0.60399	0.63690	0.66794	0.69727	0.72508	0.75150	0.77666
8	0.48391	0.51935	0.55267	0.58411	0.61385	0.64205	0.66885
9	0.37047	0.40860	0.44436	0.47801	0.50977	0.53982	0.56834
10	0.26163	0.30268	0.34105	0.37706	0.41096	0.44298	0.47329
11	0.15583	0.20006	0.24128	0.27983	0.31603	0.35013	0.38235
12	0.05176	0.09953	0.14387	0.18520	0.22389	0.26023	0.29449
13		0.0	0.04781	0.09220	0.13361	0.17240	0.20885
14				0.0	0.04442	0.08588	0.12473
15						0.0	0.04148

EXHIBIT 18-2 EXPECTED VALUES OF NORMAL ORDER STATISTICS

K/N	31	32	33	34	35	36	37
1	2.05646	2.06967	2.08241	2.09471	2.10661	2.11812	2.12928
2	1.63166	1.64712	1.66200	1.67636	1.69023	1.70362	1.71659
3	1.38268	1.39985	1.41637	1.43228	1.44762	1.46244	1.47676
4	1.19803	1.21672	1.23468	1.25196	1.26860	1.28466	1.30016
5	1.04709	1.06721	1.08652	1.10509	1.12295	1.14016	1.15677
6	0.91688	0.93841	0.95905	0.97886	0.99790	1.01624	1.03390
7	0.80066	0.82359	0.84555	0.86660	0.88681	0.90625	0.92496
8	0.69438	0.71875	0.74204	0.76435	0.78574	0.80629	0.82605
9	0.59545	0.62129	0.64596	0.66954	0.69214	0.71382	0.73465
10	0.50206	0.52943	0.55552	0.58043	0.60427	0.62710	0.64902
11	0.41287	0.44185	0.46942	0.49572	0.52084	0.54488	0.56793
12	0.32686	0.35755	0.38669	0.41444	0.44091	0.46620	0.49042
13	0.24322	0.27573	0.30654	0.33582	0.36371	0.39032	0.41576
14	0.16126	0.19572	0.22832	0.25924	0.28863	0.31663	0.34336
15	0.08037	0.11695	0.15147	0.18415	0.21515	0.24463	0.27272
16	0.0	0.03890	0.07552	0.11009	0.14282	0.17388	0.20342
17			0.0	0.03663	0.07123	0.10399	0.13509
18					0.0	0.03461	0.06739
19							0.0

K/N	38	39	40	41	42	43	44
1	2.14009	2.15059	2.16078	2.17068	2.18032	2.18969	2.19882
2	1.72914	1.74131	1.75312	1.76458	1.77571	1.78654	1.79707
3	1.49061	1.50402	1.51702	1.52964	1.54188	1.55377	1.56533
4	1.31514	1.32964	1.34368	1.35728	1.37048	1.38329	1.39574
5	1.17280	1.18830	1.20330	1.21782	1.23190	1.24556	1.25881
6	1.05095	1.06741	1.08332	1.09872	1.11364	1.12810	1.14213
7	0.94300	0.96041	0.97722	0.99348	1.00922	1.02446	1.03924
8	0.84508	0.86343	0.88114	0.89825	0.91480	0.93082	0.94634
9	0.75468	0.77398	0.79259	0.81056	0.82792	0.84472	0.86097
10	0.67009	0.69035	0.70988	0.72871	0.74690	0.76448	0.78148
11	0.59005	0.61131	0.63177	0.65149	0.67052	0.68889	0.70666
12	0.51363	0.53592	0.55736	0.57799	0.59788	0.61707	0.63561
13	0.44012	0.46348	0.48591	0.50749	0.52827	0.54830	0.56763
14	0.36892	0.39340	0.41688	0.43944	0.46114	0.48204	0.50220
15	0.29954	0.32520	0.34978	0.37337	0.39604	0.41784	0.43885
16	0.23159	0.25849	0.28423	0.30890	0.33257	0.35533	0.37723
17	0.16469	0.19292	0.21988	0.24569	0.27043	0.29418	0.31701
18	0.09853	0.12817	0.15644	0.18345	0.20931	0.23411	0.25792
19	0.03280	0.06395	0.09362	0.12192	0.14897	0.17488	0.19972
20		0.0	0.03117	0.06085	0.08917	0.11625	0.14219
21				0.0	0.02969	0.05803	0.08513
22						0.0	0.02835

EXHIBIT 18-2 EXPECTED VALUES OF NORMAL ORDER STATISTICS

K/N	45	46	47	48	49	50	51
1	2.20772	2.21639	2.22486	2.23312	2.24119	2.24907	2.25678
2	1.80733	1.81732	1.82706	1.83655	1.84582	1.85487	1.86371
3	1.57658	1.58754	1.59820	1.60860	1.61874	1.62863	1.63829
4	1.40784	1.41962	1.43108	1.44224	1.45312	1.46374	1.47409
5	1.27170	1.28422	1.29641	1.30827	1.31983	1.33109	1.34207
6	1.15576	1.16899	1.18186	1.19439	1.20658	1.21846	1.23003
7	1.05358	1.06751	1.08104	1.09420	1.10701	1.11948	1.13162
8	0.96139	0.97599	0.99018	1.00396	1.01737	1.03042	1.04312
9	0.87673	0.89201	0.90684	0.92125	0.93525	0.94887	0.96213
10	0.79795	0.81391	0.82939	0.84442	0.85902	0.87321	0.88701
11	0.72385	0.74049	0.75663	0.77228	0.78748	0.80225	0.81661
12	0.65353	0.67088	0.68768	0.70397	0.71978	0.73513	0.75004
13	0.58631	0.60438	0.62186	0.63881	0.65523	0.67117	0.68666
14	0.52166	0.54046	0.55865	0.57625	0.59331	0.60986	0.62592
15	0.45912	0.47868	0.49759	0.51588	0.53360	0.55077	0.56742
16	0.39833	0.41868	0.43834	0.45734	0.47573	0.49354	0.51080
17	0.33898	0.36016	0.38060	0.40034	0.41942	0.43789	0.45578
18	0.28081	0.30285	0.32410	0.34460	0.36441	0.38357	0.40211
19	0.22358	0.24652	0.26862	0.28992	0.31049	0.33036	0.34957
20	0.16707	0.19097	0.21396	0.23610	0.25746	0.27807	0.29799
21	0.11109	0.13600	0.15993	0.18296	0.20514	0.22653	0.24719
22	0.05546	0.08144	0.10637	0.13033	0.15338	0.17559	0.19702
23	0.0	0.02712	0.05311	0.07805	0.10203	0.12511	0.14735
24			0.0	0.02599	0.05095	0.07494	0.09803
25					0.0	0.02496	0.04896
26							0.0

EXHIBIT 18-2 EXPECTED VALUES OF NORMAL ORDER STATISTICS

K/N	52	53	54	55	56	57	58
1	2.26432	2.27169	2.27891	2.28598	2.29291	2.29970	2.30635
2	1.87235	1.88080	1.88906	1.89715	1.90506	1.91282	1.92041
3	1.64773	1.65695	1.66596	1.67478	1.68340	1.69185	1.70012
4	1.48420	1.49407	1.50372	1.51315	1.52237	1.53140	1.54024
5	1.35279	1.36326	1.37348	1.38346	1.39323	1.40278	1.41212
6	1.24132	1.25234	1.26310	1.27361	1.28387	1.29391	1.30373
7	1.14347	1.15502	1.16629	1.17729	1.18804	1.19855	1.20882
8	1.05550	1.06757	1.07934	1.09083	1.10205	1.11300	1.12371
9	0.97504	0.98762	0.99988	1.01185	1.02352	1.03493	1.04607
10	0.90045	0.91354	0.92629	0.93873	0.95086	0.96271	0.97427
11	0.83058	0.84417	0.85742	0.87033	0.88292	0.89520	0.90719
12	0.76455	0.77866	0.79240	0.80578	0.81883	0.83155	0.84397
13	0.70170	0.71633	0.73057	0.74444	0.75794	0.77111	0.78396
14	0.64152	0.65668	0.67143	0.68578	0.69976	0.71337	0.72665
15	0.58358	0.59928	0.61455	0.62940	0.64385	0.65793	0.67164
16	0.52755	0.54380	0.55960	0.57495	0.58989	0.60444	0.61860
17	0.47312	0.48995	0.50629	0.52217	0.53761	0.55263	0.56725
18	0.42007	0.43749	0.45439	0.47080	0.48675	0.50226	0.51736
19	0.36818	0.38621	0.40369	0.42065	0.43713	0.45314	0.46872
20	0.31726	0.33592	0.35400	0.37154	0.38856	0.40510	0.42117
21	0.26716	0.28648	0.30518	0.32331	0.34090	0.35797	0.37456
22	0.21772	0.23772	0.25708	0.27583	0.29400	0.31163	0.32875
23	0.16880	0.18953	0.20957	0.22896	0.24774	0.26595	0.28362
24	0.12029	0.14177	0.16252	0.18259	0.20201	0.22082	0.23906
25	0.07206	0.09434	0.11584	0.13661	0.15669	0.17614	0.19498
26	0.02400	0.04712	0.06940	0.09091	0.11170	0.13180	0.15127
27		0.0	0.02312	0.04541	0.06693	0.08773	0.10785
28				0.0	0.02229	0.04382	0.06463
29						0.0	0.02153

EXHIBIT 18-2 EXPECTED VALUES OF NORMAL ORDER STATISTICS

K/N	59	60	61	62	63	64	65
1	2.31288	2.31928	2.32556	2.33173	2.33778	2.34373	2.34958
2	1.92786	1.93516	1.94232	1.94934	1.95624	1.96301	1.96965
3	1.70822	1.71616	1.72394	1.73158	1.73906	1.74641	1.75363
4	1.54889	1.55736	1.56567	1.57381	1.58180	1.58963	1.59732
5	1.42127	1.43023	1.43900	1.44760	1.45603	1.46430	1.47241
6	1.31334	1.32274	1.33195	1.34097	1.34982	1.35848	1.36698
7	1.21886	1.22869	1.23832	1.24774	1.25698	1.26603	1.27490
8	1.13419	1.14443	1.15445	1.16427	1.17388	1.18329	1.19252
9	1.05695	1.06760	1.07802	1.08821	1.09819	1.10797	1.11754
10	0.98557	0.99662	1.00742	1.01799	1.02833	1.03846	1.04838
11	0.91890	0.93034	0.94153	0.95247	0.96317	0.97365	0.98391
12	0.85609	0.86793	0.87950	0.89081	0.90187	0.91270	0.92329
13	0.79649	0.80873	0.82068	0.83237	0.84379	0.85496	0.86590
14	0.73960	0.75224	0.76459	0.77665	0.78843	0.79996	0.81123
15	0.68502	0.69807	0.71081	0.72324	0.73540	0.74727	0.75889
16	0.63241	0.64587	0.65901	0.67183	0.68436	0.69659	0.70856
17	0.58150	0.59538	0.60893	0.62214	0.63504	0.64764	0.65996
18	0.53205	0.54637	0.56033	0.57395	0.58723	0.60020	0.61288
19	0.48388	0.49864	0.51303	0.52705	0.54073	0.55408	0.56712
20	0.43681	0.45202	0.46685	0.48129	0.49537	0.50911	0.52252
21	0.39068	0.40637	0.42164	0.43652	0.45101	0.46515	0.47894
22	0.34538	0.36155	0.37729	0.39260	0.40752	0.42207	0.43625
23	0.30078	0.31745	0.33366	0.34944	0.36480	0.37976	0.39435
24	0.25677	0.27396	0.29066	0.30691	0.32272	0.33812	0.35312
25	0.21325	0.23098	0.24820	0.26494	0.28122	0.29706	0.31249
26	0.17013	0.18842	0.20618	0.22343	0.24019	0.25650	0.27237
27	0.12733	0.14621	0.16452	0.18230	0.19957	0.21636	0.23269
28	0.08476	0.10425	0.12315	0.14148	0.15927	0.17656	0.19337
29	0.04234	0.06248	0.08198	0.10089	0.11923	0.13704	0.15435
30	0.0	0.02081	0.04096	0.06047	0.07938	0.09774	0.11556
31			0.0	0.02014	0.03966	0.05858	0.07694
32					0.0	0.01952	0.03844
33							0.0

EXHIBIT 18-2 EXPECTED VALUES OF NORMAL ORDER STATISTICS

K/N	66	67	68	69	70	71	72
1	2.35532	2.36097	2.36652	2.37199	2.37736	2.38265	2.38785
2	1.97618	1.98260	1.98891	1.99510	2.00120	2.00720	2.01310
3	1.76071	1.76767	1.77451	1.78122	1.78783	1.79432	1.80071
4	1.60487	1.61228	1.61955	1.62670	1.63373	1.64063	1.64742
5	1.48036	1.48817	1.49584	1.50338	1.51078	1.51805	1.52520
6	1.37532	1.38351	1.39154	1.39942	1.40717	1.41478	1.42226
7	1.28360	1.29213	1.30051	1.30873	1.31680	1.32473	1.33252
8	1.20157	1.21044	1.21915	1.22769	1.23608	1.24431	1.25240
9	1.12693	1.13613	1.14516	1.15401	1.16270	1.17123	1.17961
10	1.05810	1.06762	1.07696	1.08612	1.09511	1.10393	1.11259
11	0.99395	1.00380	1.01345	1.02291	1.03220	1.04130	1.05024
12	0.93367	0.94383	0.95379	0.96355	0.97313	0.98252	0.99173
13	0.87660	0.88708	0.89735	0.90741	0.91728	0.92695	0.93644
14	0.82226	0.83306	0.84364	0.85400	0.86416	0.87412	0.88388
15	0.77025	0.78138	0.79226	0.80293	0.81338	0.82362	0.83366
16	0.72025	0.73170	0.74290	0.75387	0.76462	0.77514	0.78546
17	0.67200	0.68377	0.69529	0.70657	0.71761	0.72843	0.73903
18	0.62526	0.63737	0.64921	0.66080	0.67214	0.68325	0.69413
19	0.57985	0.59230	0.60447	0.61638	0.62803	0.63943	0.65060
20	0.53561	0.54841	0.56091	0.57314	0.58510	0.59681	0.60827
21	0.49240	0.50555	0.51839	0.53095	0.54323	0.55525	0.56701
22	0.45009	0.46360	0.47680	0.48969	0.50230	0.51463	0.52669
23	0.40857	0.42245	0.43601	0.44925	0.46219	0.47484	0.48721
24	0.36775	0.38201	0.39594	0.40953	0.42281	0.43579	0.44848
25	0.32753	0.34219	0.35649	0.37045	0.38408	0.39739	0.41041
26	0.28784	0.30290	0.31759	0.33192	0.34591	0.35958	0.37292
27	0.24859	0.26408	0.27917	0.29389	0.30825	0.32227	0.33596
28	0.20973	0.22565	0.24116	0.25627	0.27102	0.28540	0.29945
29	0.17118	0.18755	0.20349	0.21902	0.23416	0.24893	0.26333
30	0.13288	0.14972	0.16611	0.18207	0.19762	0.21277	0.22756
31	0.09478	0.11211	0.12896	0.14536	0.16134	0.17690	0.19208
32	0.05681	0.07465	0.09199	0.10885	0.12527	0.14125	0.15683
33	0.01893	0.03730	0.05514	0.07249	0.08936	0.10579	0.12178
34		0.0	0.01837	0.03622	0.05357	0.07045	0.08688
35				0.0	0.01785	0.03520	0.05209
36						0.0	0.01736

EXHIBIT 18-2 EXPECTED VALUES OF NORMAL ORDER STATISTICS

K/N	73	74	75	76	77	78	79
1	2.39298	2.39802	2.40299	2.40789	2.41271	2.41747	2.42215
2	2.01890	2.02462	2.03024	2.03578	2.04124	2.04662	2.05191
3	1.80699	1.81317	1.81926	1.82525	1.83115	1.83696	1.84268
4	1.65410	1.66067	1.66714	1.67350	1.67976	1.68592	1.69200
5	1.53223	1.53914	1.54594	1.55263	1.55921	1.56569	1.57207
6	1.42961	1.43684	1.44395	1.45094	1.45782	1.46459	1.47125
7	1.34017	1.34770	1.35510	1.36237	1.36953	1.37657	1.38350
8	1.26034	1.26815	1.27583	1.28338	1.29080	1.29810	1.30529
9	1.18784	1.19592	1.20387	1.21168	1.21936	1.22691	1.23434
10	1.12110	1.12945	1.13766	1.14572	1.15365	1.16145	1.16912
11	1.05902	1.06764	1.07610	1.08442	1.09260	1.10063	1.10854
12	1.00078	1.00966	1.01838	1.02695	1.03537	1.04364	1.05178
13	0.94576	0.95490	0.96387	0.97269	0.98135	0.98986	0.99822
14	0.89346	0.90286	0.91209	0.92115	0.93005	0.93880	0.94739
15	0.84351	0.85317	0.86265	0.87196	0.88110	0.89008	0.89890
16	0.79558	0.80550	0.81524	0.82480	0.83418	0.84339	0.85244
17	0.74942	0.75960	0.76960	0.77940	0.78903	0.79848	0.80776
18	0.70480	0.71526	0.72551	0.73557	0.74544	0.75512	0.76463
19	0.66155	0.67227	0.68279	0.69310	0.70322	0.71314	0.72289
20	0.61950	0.63050	0.64128	0.65185	0.66222	0.67239	0.68237
21	0.57852	0.58980	0.60085	0.61168	0.62230	0.63272	0.64294
22	0.53850	0.55006	0.56138	0.57248	0.58336	0.59403	0.60449
23	0.49932	0.51117	0.52277	0.53414	0.54528	0.55621	0.56692
24	0.46089	0.47304	0.48493	0.49657	0.50798	0.51917	0.53013
25	0.42313	0.43558	0.44777	0.45970	0.47138	0.48283	0.49404
26	0.38597	0.39873	0.41122	0.42343	0.43540	0.44711	0.45859
27	0.34934	0.36242	0.37521	0.38772	0.39997	0.41196	0.42371
28	0.31317	0.32657	0.33968	0.35250	0.36504	0.37731	0.38934
29	0.27740	0.29114	0.30457	0.31770	0.33055	0.34311	0.35542
30	0.24199	0.25608	0.26984	0.28329	0.29645	0.30931	0.32190
31	0.20688	0.22133	0.23543	0.24922	0.26269	0.27586	0.28875
32	0.17202	0.18684	0.20130	0.21543	0.22923	0.24272	0.25591
33	0.13737	0.15257	0.16740	0.18188	0.19602	0.20983	0.22334
34	0.10289	0.11848	0.13370	0.14854	0.16303	0.17718	0.19101
35	0.06852	0.08453	0.10014	0.11536	0.13021	0.14471	0.15888
36	0.03424	0.05068	0.06670	0.08231	0.09754	0.11240	0.12691
37	0.0	0.01689	0.03333	0.04935	0.06497	0.08020	0.09507
38			0.0	0.01644	0.03247	0.04809	0.06333
39					0.0	0.01602	0.03165
40							0.0

EXHIBIT 18-2 EXPECTED VALUES OF NORMAL ORDER STATISTICS

K/N	80	81	82	83	84	85	86
1	2.42677	2.43133	2.43582	2.44026	2.44463	2.44894	2.45320
2	2.05714	2.06228	2.06735	2.07236	2.07729	2.08216	2.08696
3	1.84832	1.85387	1.85935	1.86475	1.87007	1.87532	1.88049
4	1.69798	1.70387	1.70968	1.71540	1.72104	1.72660	1.73209
5	1.57836	1.58455	1.59065	1.59665	1.60258	1.60841	1.61417
6	1.47781	1.48428	1.49064	1.49691	1.50309	1.50918	1.51518
7	1.39032	1.39704	1.40366	1.41017	1.41659	1.42292	1.42915
8	1.31236	1.31932	1.32617	1.33292	1.33957	1.34611	1.35257
9	1.24165	1.24884	1.25593	1.26290	1.26977	1.27653	1.28320
10	1.17666	1.18409	1.19139	1.19859	1.20567	1.21264	1.21951
11	1.11631	1.12396	1.13148	1.13889	1.14618	1.15336	1.16043
12	1.05978	1.06764	1.07539	1.08300	1.09050	1.09788	1.10515
13	1.00644	1.01453	1.02249	1.03031	1.03802	1.04560	1.05306
14	0.95584	0.96414	0.97231	0.98034	0.98825	0.99603	1.00369
15	0.90757	0.91609	0.92447	0.93271	0.94082	0.94880	0.95665
16	0.86134	0.87007	0.87867	0.88711	0.89542	0.90360	0.91164
17	0.81687	0.82583	0.83464	0.84329	0.85180	0.86017	0.86841
18	0.77398	0.78315	0.79217	0.80103	0.80975	0.81832	0.82675
19	0.73246	0.74186	0.75109	0.76016	0.76908	0.77785	0.78647
20	0.69217	0.70179	0.71124	0.72053	0.72965	0.73862	0.74744
21	0.65297	0.66282	0.67249	0.68199	0.69133	0.70050	0.70952
22	0.61476	0.62484	0.63473	0.64445	0.65399	0.66337	0.67259
23	0.57742	0.58773	0.59785	0.60779	0.61755	0.62714	0.63656
24	0.54088	0.55143	0.56178	0.57193	0.58191	0.59171	0.60133
25	0.50504	0.51583	0.52641	0.53680	0.54700	0.55701	0.56684
26	0.46985	0.48088	0.49170	0.50232	0.51274	0.52297	0.53301
27	0.43522	0.44651	0.45757	0.46842	0.47907	0.48952	0.49979
28	0.40111	0.41265	0.42397	0.43506	0.44594	0.45662	0.46710
29	0.36747	0.37927	0.39084	0.40218	0.41330	0.42421	0.43491
30	0.33423	0.34630	0.35813	0.36972	0.38108	0.39223	0.40316
31	0.30136	0.31371	0.32580	0.33765	0.34926	0.36065	0.37182
32	0.26881	0.28144	0.29381	0.30592	0.31779	0.32943	0.34084
33	0.23655	0.24947	0.26212	0.27450	0.28664	0.29852	0.31018
34	0.20453	0.21775	0.23069	0.24335	0.25576	0.26790	0.27981
35	0.17272	0.18625	0.19949	0.21244	0.22512	0.23753	0.24970
36	0.14108	0.15493	0.16848	0.18172	0.19469	0.20738	0.21981
37	0.10959	0.12377	0.13763	0.15118	0.16444	0.17741	0.19012
38	0.07820	0.09272	0.10691	0.12078	0.13434	0.14761	0.16059
39	0.04689	0.06177	0.07629	0.09049	0.10436	0.11793	0.13121
40	0.01562	0.03087	0.04575	0.06028	0.07448	0.08836	0.10193
41		0.0	0.01524	0.03013	0.04466	0.05886	0.07275
42				0.0	0.01488	0.02942	0.04362
43						0.0	0.01454

EXHIBIT 18-2 EXPECTED VALUES OF NORMAL ORDER STATISTICS

K/N	87	88	89	90	91	92	93
1	2.45741	2.46156	2.46565	2.46970	2.47370	2.47764	2.48154
2	2.09170	2.09637	2.10099	2.10554	2.11004	2.11448	2.11887
3	1.88560	1.89064	1.89561	1.90052	1.90536	1.91015	1.91487
4	1.73750	1.74283	1.74810	1.75329	1.75842	1.76348	1.76848
5	1.61984	1.62544	1.63096	1.63641	1.64178	1.64709	1.65232
6	1.52110	1.52693	1.53269	1.53836	1.54396	1.54949	1.55494
7	1.43529	1.44135	1.44732	1.45321	1.45903	1.46476	1.47042
8	1.35893	1.36520	1.37138	1.37747	1.38348	1.38941	1.39526
9	1.28976	1.29624	1.30262	1.30891	1.31511	1.32123	1.32726
10	1.22628	1.23295	1.23952	1.24600	1.25239	1.25869	1.26491
11	1.16740	1.17426	1.18102	1.18769	1.19426	1.20073	1.20712
12	1.11231	1.11936	1.12631	1.13316	1.13990	1.14656	1.15311
13	1.06041	1.06765	1.07478	1.08181	1.08873	1.09555	1.10228
14	1.01122	1.01865	1.02596	1.03316	1.04026	1.04726	1.05415
15	0.96437	0.97198	0.97948	0.98686	0.99413	1.00129	1.00835
16	0.91956	0.92735	0.93502	0.94258	0.95002	0.95735	0.96458
17	0.87651	0.88449	0.89234	0.90007	0.90769	0.91519	0.92258
18	0.83504	0.84320	0.85123	0.85914	0.86693	0.87460	0.88215
19	0.79496	0.80330	0.81152	0.81960	0.82756	0.83540	0.84312
20	0.75611	0.76465	0.77304	0.78131	0.78944	0.79745	0.80533
21	0.71838	0.72710	0.73568	0.74412	0.75243	0.76061	0.76866
22	0.68165	0.69056	0.69932	0.70795	0.71643	0.72478	0.73300
23	0.64581	0.65492	0.66387	0.67267	0.68134	0.68986	0.69825
24	0.61079	0.62009	0.62923	0.63822	0.64706	0.65576	0.66432
25	0.57650	0.58600	0.59533	0.60451	0.61353	0.62241	0.63115
26	0.54288	0.55258	0.56210	0.57147	0.58068	0.58974	0.59865
27	0.50986	0.51976	0.52949	0.53905	0.54845	0.55769	0.56678
28	0.47739	0.48750	0.49743	0.50718	0.51677	0.52620	0.53547
29	0.44542	0.45574	0.46587	0.47582	0.48561	0.49522	0.50468
30	0.41389	0.42443	0.43477	0.44493	0.45491	0.46472	0.47436
31	0.38278	0.39353	0.40409	0.41445	0.42463	0.43464	0.44447
32	0.35203	0.36300	0.37378	0.38436	0.39474	0.40495	0.41498
33	0.32161	0.33281	0.34381	0.35461	0.36520	0.37561	0.38584
34	0.29148	0.30292	0.31415	0.32517	0.33598	0.34660	0.35702
35	0.26162	0.27330	0.28476	0.29601	0.30704	0.31787	0.32850
36	0.23199	0.24392	0.25562	0.26710	0.27835	0.28940	0.30025
37	0.20256	0.21475	0.22669	0.23841	0.24990	0.26117	0.27223
38	0.17330	0.18576	0.19796	0.20991	0.22164	0.23314	0.24443
39	0.14420	0.15692	0.16938	0.18159	0.19356	0.20530	0.21681
40	0.11521	0.12821	0.14094	0.15341	0.16563	0.17761	0.18936
41	0.08633	0.09961	0.11262	0.12536	0.13783	0.15006	0.16205
42	0.05751	0.07110	0.08439	0.09740	0.11014	0.12262	0.13486
43	0.02874	0.04263	0.05622	0.06952	0.08253	0.09528	0.10777
44	0.0	0.01421	0.02810	0.04169	0.05499	0.06801	0.08076
45			0.0	0.01389	0.02748	0.04078	0.05381
46					0.0	0.01359	0.02689
47							0.0

EXHIBIT 18-2 EXPECTED VALUES OF NORMAL ORDER STATISTICS

K/N	94	95	96	97	98	99	100
1	2.48540	2.48920	2.49297	2.49669	2.50036	2.50400	2.50759
2	2.12321	2.12749	2.13172	2.13590	2.14003	2.14411	2.14814
3	1.91953	1.92414	1.92869	1.93318	1.93763	1.94201	1.94635
4	1.77341	1.77828	1.78309	1.78784	1.79254	1.79718	1.80176
5	1.65749	1.66259	1.66763	1.67261	1.67752	1.68238	1.68718
6	1.56033	1.56564	1.57089	1.57607	1.58118	1.58624	1.59123
7	1.47600	1.48151	1.48695	1.49232	1.49762	1.50286	1.50803
8	1.40103	1.40673	1.41235	1.41790	1.42338	1.42879	1.43414
9	1.33321	1.33909	1.34489	1.35061	1.35626	1.36183	1.36734
10	1.27104	1.27708	1.28305	1.28894	1.29475	1.30049	1.30615
11	1.21342	1.21964	1.22577	1.23182	1.23779	1.24368	1.24950
12	1.15958	1.16596	1.17226	1.17847	1.18459	1.19064	1.19661
13	1.10891	1.11546	1.12191	1.12827	1.13455	1.14075	1.14687
14	1.06095	1.06765	1.07426	1.08078	1.08721	1.09356	1.09982
15	1.01531	1.02217	1.02894	1.03561	1.04219	1.04868	1.05509
16	0.97170	0.97872	0.98564	0.99246	0.99919	1.00583	1.01238
17	0.92986	0.93704	0.94411	0.95109	0.95797	0.96475	0.97145
18	0.88959	0.89693	0.90416	0.91129	0.91831	0.92524	0.93208
19	0.85072	0.85822	0.86560	0.87288	0.88006	0.88713	0.89411
20	0.81310	0.82075	0.82829	0.83572	0.84305	0.85027	0.85739
21	0.77659	0.78441	0.79210	0.79968	0.80716	0.81452	0.82179
22	0.74110	0.74907	0.75692	0.76466	0.77228	0.77980	0.78720
23	0.70651	0.71464	0.72266	0.73055	0.73832	0.74598	0.75353
24	0.67275	0.68105	0.68922	0.69727	0.70519	0.71301	0.72070
25	0.63974	0.64821	0.65654	0.66474	0.67282	0.68079	0.68863
26	0.60742	0.61605	0.62454	0.63291	0.64115	0.64926	0.65725
27	0.57572	0.58452	0.59318	0.60170	0.61010	0.61837	0.62651
28	0.54459	0.55356	0.56239	0.57108	0.57963	0.58805	0.59635
29	0.51398	0.52312	0.53212	0.54097	0.54969	0.55827	0.56672
30	0.48384	0.49316	0.50233	0.51136	0.52024	0.52898	0.53758
31	0.45414	0.46364	0.47299	0.48218	0.49123	0.50013	0.50890
32	0.42483	0.43452	0.44404	0.45341	0.46263	0.47170	0.48062
33	0.39588	0.40576	0.41547	0.42501	0.43440	0.44364	0.45273
34	0.36727	0.37733	0.38722	0.39695	0.40652	0.41593	0.42518
35	0.33895	0.34921	0.35929	0.36920	0.37895	0.38853	0.39796
36	0.31090	0.32136	0.33163	0.34173	0.35166	0.36142	0.37102
37	0.28309	0.29375	0.30423	0.31452	0.32464	0.33458	0.34436
38	0.25550	0.26637	0.27705	0.28754	0.29785	0.30797	0.31793
39	0.22810	0.23919	0.25008	0.26077	0.27127	0.28159	0.29173
40	0.20088	0.21219	0.22328	0.23418	0.24488	0.25539	0.26572
41	0.17380	0.18533	0.19665	0.20776	0.21866	0.22937	0.23990
42	0.14685	0.15861	0.17015	0.18148	0.19259	0.20351	0.21423
43	0.12001	0.13201	0.14378	0.15533	0.16666	0.17778	0.18870
44	0.09325	0.10550	0.11750	0.12928	0.14083	0.15217	0.16330
45	0.06656	0.07906	0.09131	0.10332	0.11510	0.12666	0.13800
46	0.03992	0.05267	0.06518	0.07743	0.08944	0.10123	0.11279
47	0.01330	0.02633	0.03909	0.05159	0.06385	0.07586	0.08765
48		0.0	0.01303	0.02579	0.03829	0.05055	0.06257
49				0.0	0.01276	0.02527	0.03753
50						0.0	0.01251

Chapter 19
Transmission Losses

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Chapter 19
Transmission Losses

Introduction

Streams in natural channels in arid and semiarid regions are usually ephemeral. Flow is occasional and follows storms, which are infrequent. When flood flows occur in normally dry stream channels, the volume of flow is reduced by infiltration into the bed, the banks, and possibly the flood plain. These losses to infiltration, called transmission losses, reduce not only the volume of the hydrograph, but also the peak discharge.

This chapter describes a procedure for estimating the volume of runoff and peak discharge for ephemeral streams; it can be used with or without observed inflow-outflow data. If available, observed inflow-outflow data can be used to derive regression equations for the particular channel reach. Procedures based on the derived regression equations enable a user to determine prediction equations for similar channels of arbitrary length and width.

Also presented are procedures for estimating parameters of the prediction equations in the absence of observed inflow-outflow data. These procedures are based on characteristics of the bed and bank material. Approximations for lateral inflow and out-of-bank flow are also presented.

Assumptions and Limitations

Assumptions

The methods described in this chapter are based on the following assumptions:

1. Water is lost in the channel; no streams gain water.
2. Infiltration characteristics and other channel properties are uniform with distance and width.
3. Sediment concentration, temperature, and antecedent flow affect transmission losses, but the equations represent the average conditions.
4. The channel reach is short enough that an average width and an average duration represent the width and duration of flow for the entire channel reach.
5. Once a threshold volume has been satisfied, outflow volumes are linear with inflow volumes.
6. Once an average loss rate is subtracted and the inflow volume exceeds the threshold volume, peak rates of outflow are linear with peak rates of inflow. Moreover, the rate of change in outflow peak discharge with changing inflow peak discharge is the same as the rate of change in outflow volume with changing inflow volume.

7. Lateral inflow can be either lumped at points of tributary inflow or uniform with distance along the channel.

8. For volume and peak discharge calculations, lateral inflow is assumed to occur during the same time as the upstream inflow.

Limitations

The main limitations of the procedures are:

1. Hydrographs are not specifically routed along the stream channels; predictions are made for volume and peak discharge.
2. Peak flow equations do not consider storage attenuation effects or steepening of the hydrograph rise.
3. Analyses on which the procedures are based represent average conditions or overall trends.
4. Influences of antecedent flow and sediment concentration in the streamflow have not been quantified.
5. Estimates of effective hydraulic conductivity in the streambed are empirically based and represent average rates.
6. Peak discharge of outflow is decreased by the average loss rate for the duration of flow.
7. Procedures for out-of-bank flow are based on the assumption of a weighted average for the effective hydraulic conductivity.

Symbols and Notation

Upstream Inflow

D = duration of inflow (hours)
P = inflow volume (acre-feet)
p = peak rate of inflow (cubic feet per second)

Lateral Inflow

Q_L = lateral inflow volume (acre-feet per mile)
q_L = peak rate of lateral inflow (cubic feet per second per foot)

Outflow

Q(x,w) = outflow volume (acre-feet)
q(x,w) = peak rate of outflow (cubic feet per second)

Channel Reach

D = duration of streamflow (hours)
K = effective hydraulic conductivity (inches per hour)
V = total available storage volume of alluvium in the channel reach (acre-feet)
w = average width of flow (feet)
x = length of reach (miles)

Prediction Equations (Parameters)

a = regression intercept for unit channel (acre-feet)
a(D) = regression intercept for unit channel with a flow of duration D (acre-feet)
a(x,w) = regression intercept for a channel reach of length x and width w (acre-feet)
b = regression slope for unit channel
b(x,w) = regression slope for a channel reach of length x and width w
k = decay factor $(\text{foot-miles})^{-1}$
k(D,P) = decay factor for unit channel with a flow duration D and volume P $(\text{foot-miles})^{-1}$
P_o = threshold volume for a unit channel (acre-feet)
$P_o(x,w)$ = threshold volume for a channel reach of length x and width w (acre-feet)

The simplified procedures are summarized here; additional details and derivations are given in the appendices. Methods have been developed for two situations: (1) when observed inflow-outflow data are available and (2) when no observed data are available.

Summary of Procedure

The prediction equation for outflow volume, without lateral inflow, is

$$Q(x,w) = \begin{cases} 0 & P \leq P_o(x,w) \\ a(x,w) + b(x,w)P & P > P_o(x,w), \end{cases} \tag{19–1}$$

where the threshold volume is

$$P_o(x,w) = \frac{-a(x,w)}{b(x,w)}. \tag{19–2}$$

The corresponding equation for peak discharge is

$$q(x,w) = \begin{cases} 0 & Q(x,w) = 0 \\ \frac{12.1}{D}(a(x,w) - [1 - b(x,w)]P) + b(x,w)p & Q(x,w) > 0, \end{cases} \tag{19–3}$$

where 12.1 converts from acre-feet per hour to cubic feet per second.

If lateral inflow is uniform, the volume equation becomes

$$Q(x,w) = \begin{cases} 0 & b(x,w)P + \frac{Q_L}{kw}[1 - b(x,w)] \leq -a(x,w) \\ a(x,w) + b(x,w)P + \frac{Q_L}{kw}[1 - b(x,w)]. \end{cases} \tag{19–4}$$

The corresponding equation for peak discharge is

$$q(x,w) = \begin{cases} 0 & Q(x,w) = 0 \\ \frac{12.1}{D}(a(x,w) - [1 - b(x,w)]P) + b(x,w)p + \frac{q_L(5{,}280)}{kw}[1 - b(x,w)]. \end{cases} \tag{19–5}$$

The factor 5,280 converts cubic feet per second per foot to cubic feet per second per mile. Derivations and background information are found in Appendix 1.

For a channel reach with only tributary lateral inflow, equations 19–1 and 19–3 would be applied on the tributary channel and the main channel to the point of tributary inflow. Then the sum of the outflows from these two channel reaches would be the inflow to the lower reach of the main channel.

The procedures described by equations 19–1, 19–3, 19–4, and 19–5 require that the upstream inflow and lateral inflow along the channel reach be estimated by use of procedures described in Chapter 10. Peak rates and durations are estimated by use of procedures described in Chapter 16.

Estimating Parameters From Observed Inflow-Outflow Data

If one assumes a channel reach of length x and average width w, then n observations on P_i and Q_i (without lateral inflow) can be used to estimate the parameters in equation 19–1. Parameters of the linear regression equation can be estimated as

$$b(x,w) = \frac{\sum_{i=1}^{n} (Q_i - \overline{Q})(P_i - \overline{P})}{\sum_{i=1}^{n} (P_i - \overline{P})^2} \tag{19–6}$$

and

$$a(x,w) = \overline{Q} - b(x,w)\overline{P}, \tag{19–7}$$

where $\overline{Q}$ is the mean outflow volume and $\overline{P}$ is the mean inflow volume. Alternative formulas recommended for computation are

$$\sum_{i=1}^{n} (Q_i - \overline{Q})(P_i - \overline{P}) = \frac{n\sum_{i=1}^{n} P_iQ_i - \left(\sum_{i=1}^{n} P_i\right)\left(\sum_{i=1}^{n} Q_i\right)}{n} \tag{19–8}$$

and

$$\sum_{i=1}^{n} (P_i - \overline{P})^2 = \frac{n\sum_{i=1}^{n} P_i^2 - \left(\sum_{i=1}^{n} P_i\right)^2}{n}. \tag{19–9}$$

Linear regression procedures are available on most computer systems and on many hand-held calculators. Constraints on the parameters are

$$a(x,w) < 0$$

and

$$0 \leq b(x,w) \leq 1.$$

When one or both of the constraints are not met, the following procedure is suggested:

1. Plot the observed data on rectangular coordinate paper: P_i on the X-axis and Q_i on the Y-axis.
2. Plot the derived regression equation on the graph with the data.
3. Check the data for errors (events with lateral inflow, computational errors, etc.). Pay particular attention to any data points very far from the regression line, especially those points that may be strongly influencing the slope or intercept.
4. Correct data points that are in error; remove points that are not representative.
5. Recompute the regression slope and intercept using equations 19–6 to 19–9 and the corrected data.

A great deal of care and engineering judgment must be exercised in finding and eliminating errors from the set of observed inflow-outflow observations.

Unit Channels

A unit channel is defined as a channel of length x = 1 mi and width w = 1 ft. Parameters for the unit channel are required to compute parameters for channel reaches with arbitrary length and width. The unit channel parameters are computed by the following equations:

$$k = -\frac{\ln b(x,w)}{x\,w} \qquad \textbf{(19–10)}$$

$$b = e^{-k} \qquad \textbf{(19–11)}$$

$$a = \frac{a(x,w)(1 - b)}{[1 - b(x,w)]}, \qquad \textbf{(19–12)}$$

where a(x,w) and b(x,w) are the regression parameters derived from the observed data. In this case, the length x and width w are fixed known values. Particular care must be taken to maintain the maximum number of significant digits in determining k, b, and a. Otherwise, significant round-off errors can result.

Reaches of Arbitrary Length and Width

Given parameters for a unit channel, parameters for a channel reach of arbitrary length x and arbitrary width w are computed by the following equations:

$$b(x,w) = e^{-kxw}, \qquad \textbf{(19–13)}$$

$$a(x,w) = \frac{a}{1 - b}[1 - b(x,w)], \qquad \textbf{(19–14)}$$

$$P_o(x,w) = \frac{-a(x,w)}{b(x,w)}. \qquad \textbf{(19–2)}$$

Estimating Parameters in the Absence of Observed Inflow-Outflow Data

When inflow-outflow data are not available, an estimate of effective hydraulic conductivity is needed to predict transmission losses. Effective hydraulic conductivity, K, is the infiltration rate averaged over the total area wetted by the flow and over the total duration of flow. Because effective hydraulic conductivity represents a space-time average infiltration rate, it incorporates the influence of temperature, sediment concentration, flow irregularities, errors in the data, and variations in wetted area. For this reason, it is not the same as the saturated hydraulic conductivity for clear water under steady-state conditions.

Analysis of observed data resulted in equations of the form

$$a(D) = -0.00465KD \qquad \textbf{(19–15)}$$

for the unit channel intercept and

$$k(D,P) = -1.09 \ln\left[1.0 - 0.0545\frac{KD}{P}\right] \qquad \textbf{(19–16)}$$

for the decay factor on ungaged reaches. Given values of a and k from equations 19–15 and 19–16, equations 19–13, 19–14, and 19–2 are used to compute parameters for a particular x and w.

Derived relationships between bed material characteristics, effective hydraulic conductivity, and the unit channel parameters a and k are shown in table 19–1. These data can be used to estimate parameters for ungaged channel reaches.

Table 19–1.—Relationships between bed material characteristics and parameters for a unit channel (average antecedent conditions)

Bed material group	Bed material characteristics	Effective hydraulic conductivity,[1] K	Unit channel parameters	
			Intercept,[2] a	Decay factor,[3] k
		in/hr	*acre-ft*	*(ft-mi)$^{-1}$*
1 Very high loss rate	Very clean gravel and large sand	>5	< −0.023	>0.030
2 High loss rate	Clean sand and gravel, field conditions	2.0–5.0	−0.0093 to −0.023	0.0120 to 0.030
3 Moderately high loss rate	Sand and gravel mixture with low silt-clay content	1.0–3.0	−0.0047 to −0.014	0.0060 to 0.018
4 Moderate loss rate	Sand and gravel mixture with high silt-clay content	0.25–1.0	−0.0012 to −0.0047	0.0015 to 0.0060
5 Insignificant to low loss rate	Consolidated bed material; high silt-clay content	0.001–0.10	-5×10^{-6} to -5×10^{-4}	6×10^{-6} to 6×10^{-4}

[1] See Appendix 3 for sources of basic data.
[2] Values are for unit duration, D = 1 hr. For other durations, a(D) = −0.00465KD.
[3] Values are for unit duration and volume, D/P = 1. For other durations and volumes,

use $k(D,P) = -1.09 \ln \left[1.0 - 0.00545\frac{KD}{P}\right]$.

Summary of Parameter Estimation Techniques

Suggested procedures for use when observed data are available are summarized in table 19–2. Procedures for use on ungaged channel reaches are summarized in table 19–3. Again, whatever procedure is used, the parameter estimates must satisfy the constraints $a(x,w) < 0$ and $0 \leq b(x,w) \leq 1$.

Table 19–2.—Procedures to use when observed inflow-outflow data are available

Step	Source	Result
1. Perform regression analysis	Eqs. 19–6, 19–7, 19–2	Prediction equations for the particular reach
2. Derive unit channel parameters	Eqs. 19–10 to 19–12	Unit channel parameters
3. Calculate parameters	Eqs. 19–13, 19–14, 19–2	Parameters of the prediction equations for arbitrary x and w

Table 19–3.—Procedures to use when no observed inflow-outflow data are available

Step	Source	Result
1. Estimate inflow	Hydrologic analysis	Mean duration of flow, D, and volume of inflow, P
2. Identify bed material	Table 19–1	Effective hydraulic conductivity, K
3. Derive unit channel parameters	Eqs. 19–15, 19–16, 19–11	Unit channel parameters
4. Calculate parameters	Eqs. 19–13, 19–14, 19–2	Parameters of the prediction equations for arbitrary x and w

The following examples illustrate application of the procedures for several cases under a variety of circumstances. As in any analysis, it was impossible to consider all possible combinations of circumstances, but the examples presented here should provide an overview of useful applications of the procedures. Use of these procedures requires judgment and experience. At each step of the process, care should be taken to ensure that the results are reasonable and consistent with sound engineering practice.

Example 1. No Lateral Inflow or Out-of-Bank Flow

Given: A channel reach of length x = 5.0 mi, of average width w = 70 ft, and with bed material consisting of sand and gravel with a small percentage of silt and clay. Assume a mean flow duration D = 4 hr and a mean inflow volume of P = 34 acre-ft.

Find: The prediction equations for the channel reach. Estimate the outflow volume and peak for an inflow P = 50 acre-ft and p = 1,000 cfs.

Case 1. Observed Inflow-Outflow Data

Observed Inflow-Outflow Data (acre-ft)

P_i	20.	100.	25.	10.	15.	$\overline{P} = 34$
Q_i	6.0	75.	9.0	0.1	2.5	$\overline{Q} = 18.52$

Solution: Follow the procedure outlined in table 19–2, Step 1, for x = 5.0 mi and w = 70 ft.

$$b(x,w) = \frac{\Sigma(Q_i - \overline{Q})(P_i - \overline{P})}{\Sigma(P_i - \overline{P})^2} = 0.850$$

$$\begin{aligned} a(x,w) &= \overline{Q} - b(x,w)\overline{P} \\ &= 18.52 - 0.850(34) = -10.38 \text{ acre-ft} \end{aligned}$$

$$P_o(x,w) = \frac{-a(x,w)}{b(x,w)} = \frac{10.38}{0.850} = 12.21 \text{ acre-ft}$$

Substituting these values in equation 19–1, the prediction equation for volume is

$$Q(x,w) = \begin{cases} 0 & P \leq 12.21 \\ -10.38 + 0.850P & P > 12.21 \end{cases}$$

and the prediction equation (from equation 19–3) for peak discharge is

$$q(x,w) = \begin{cases} 0 & Q(x,w) = 0 \\ -31.4 - 0.454P \\ \quad + 0.850p & Q(x,w) > 0. \end{cases}$$

For an inflow volume P = 50 acre-ft and an inflow peak rate p = 1,000 cfs, the predicted outflow volume is

$$Q(x,w) = -10.38 + 0.850(50) = 32.1 \text{ acre-ft}$$

and the predicted peak rate of outflow is

$$\begin{aligned} q(x,w) &= -31.4 - 0.454(50) + 0.850(1{,}000) \\ &= 796 \text{ cfs.} \end{aligned}$$

Case 2. No Observed Inflow-Outflow Data

Solution: Follow the procedures outlined in table 19–3.

From table 19–1, estimate K = 1.0 in/hr, with D = 4.0 hr, P = 34 acre-ft. So

$$a = -0.00465KD = -0.01860 \text{ acre-ft},$$

$$\begin{aligned} k &= -1.09 \ln\left[1.0 - 0.00545 \frac{KD}{P}\right] \\ &= 0.000699 \text{ (ft-mi)}^{-1}, \end{aligned}$$

and

$$b = e^{-k} = e^{-0.000699} = 0.999301$$

are the unit channel parameters. From equations 19–13, 19–14, and 19–2, the parameters for the given reach with x = 5.0 mi and w = 70 ft are

$$\begin{aligned} b(x,w) &= e^{-kxw} = e^{-(0.000699)(5.0)(70)} \\ &= 0.783, \end{aligned}$$

$$\begin{aligned} a(x,w) &= \frac{a}{1-b}[1 - b(x,w)] \\ &= \frac{-0.01860}{(1 - 0.999301)}[1 - 0.783] \\ &= -5.78 \text{ acre-ft}, \end{aligned}$$

and

$$
\begin{aligned}
P_o(x,w) &= \frac{-a(x,w)}{b(x,w)} \\
&= -\frac{(-5.78)}{0.783} = 7.38 \text{ acre-ft.}
\end{aligned}
$$

The prediction equation for the volume is

$$
Q(x,w) = \begin{cases} 0 & P < 7.38 \\ -5.78 + 0.783P, & P > 7.38 \end{cases}
$$

and the prediction equation for peak discharge is

$$
q(x,w) = \begin{cases} 0 & Q(x,w) = 0 \\ -17.5 - 0.656P & \\ \quad + 0.783p & Q(x,w) > 0. \end{cases}
$$

For an inflow volume of P = 50 acre-ft and an inflow peak rate of p = 1,000 cfs, the predicted outflow volume is

$$
Q(x,w) = -5.78 + 0.783(50) = 33.4 \text{ acre-ft,}
$$

and the predicted peak rate of outflow is

$$
\begin{aligned}
q(x,w) &= -17.5 - 0.656(50) + 0.783(1{,}000) \\
&= 733 \text{ cfs.}
\end{aligned}
$$

This example illustrates application of the procedures with and without observed data when flow is within the channel banks and there is no lateral inflow. The next example is for the same channel reach but is based on assumption of uniform lateral inflow between the inflow and outflow stations.

Example 2. Uniform Lateral Inflow

Given: The channel reach parameters from Example 1 and a lateral inflow of 21.3 acre-ft at a peak rate of 500 cfs. Assume the lateral inflow is uniformly distributed.

Find: The volume of outflow and peak rate of outflow if P = 50 acre-ft and p = 1,000 cfs.

Solution: Compute the lateral rates as follows:

$$
Q_L = \frac{21.3 \text{ acre-ft}}{5.0 \text{ mi}} = 4.26 \text{ acre-ft/mi}
$$

and

$$
q_L = \frac{500 \text{ cfs}}{(5.0 \text{ mi})(5{,}280 \text{ ft/mi})} = 0.0189 \text{ cfs/ft.}
$$

Using a(x,w) = −5.78, b(x,w) = 0.783, k = 0.000699, and w = 70 from Case 2 of Example 1 in equation 19–4, the result is

$$
\begin{aligned}
Q(x,w) &= -5.78 + 0.783P + \frac{Q_L}{kw}(1 - 0.783) \\
&= 52.3 \text{ acre-ft.}
\end{aligned}
$$

The corresponding calculations for peak discharge of the outflow hydrograph (eq. 19–5) are

$$
\begin{aligned}
q(x,w) &= -17.5 - 0.656P + 0.783p \\
&\quad + \frac{q_L\,(5{,}280)}{kw}[1 - 0.783] \\
&= 1{,}175 \text{ cfs.}
\end{aligned}
$$

Example 3. Approximations for Out-of-Bank Flow

In this example, approximations for out-of-bank flow are described and discussed.

Given: A channel reach of length x = 10 mi and an average width of in-bank flow w_1 = 150 ft with in-bank flow up to a discharge of 3,000 cfs. Once the flow exceeds 3,000 cfs, out-of-bank flow rapidly covers wide areas. The bed material consists of clean sand and gravel, and the out-of-bank material is sandy with significant amounts of silt-clay.

Find: The outflow if the inflow is P = 700 acre-ft with a peak rate of p = 4,000 cfs. Assume the mean duration of flow is 12 hr and the total average width of out-of-bank flow is 400 ft. Also, estimate the distance downstream before the flow is back within the channel banks.

Solution: Using the procedures outlined in table 19–3, make the following calculations:

In-bank flow:

$w_1 = 150$ ft;

$K_1^* = 3.0$ in/hr.

Out-of-bank flow:

$w_2 = 400$ ft†;

$K_2^* = 0.5$ in/hr for width $w_2 - w_1$.

The weighted average for effective hydraulic conductivity is

$$K = \frac{w_1K_1 + (w_2 - w_1)K_2}{w_2} \qquad (19\text{–}17)$$

$$K = 1.44 \text{ in/hr}.$$

Using this average value of K, D = 12 hr, and P = 700 acre-ft, the unit channel parameters are

$$a = -0.00465KD = -0.08035 \text{ acre-ft},$$

$$k = -1.09 \ln\left[1.0 - 0.00545\frac{KD}{P}\right] = 0.000147 \text{ (ft-mi)}^{-1},$$

and

$$b = e^{-k} = e^{-0.000147} = 0.99985.$$

Given the unit channel parameters and $w_2 = 400$ ft, the parameters for the channel reach are

$$b(x,w_2) = e^{-kxw_2} = e^{-(0.000147)(400)x} = e^{-0.0588x}$$

and

$$a(x,w_2) = \frac{a}{1-b}[1 - b(x,w_2)] = \frac{-0.08035}{(1 - 0.99985)}[1 - e^{-0.0588x}].$$

Now, estimate the distance downstream until flow is contained within the banks (from equation 19–3) as

$$q(x,w) = \frac{12.1}{D}(a(x,w) - [1 - b(x,w)]P) + b(x,w)p.$$

Use an upper limit as

$$q(x,w) = 3{,}000 \text{ cfs} \leqslant b(x,w)p = e^{-0.0588x}(4{,}000),$$

which means

$$e^{-0.0588x} \geqslant \frac{3{,}000}{4{,}000} = 0.75$$

$$x \leqslant -\frac{1.0}{0.0588}\ln 0.75 = 4.89 \text{ mi}.$$

Then a trial-and-error solution of the volume and peak discharge equations for various values of $x < 4.89$ mi produces a best estimate of $x = 3.6$ mi. Based on this value, the parameters are

$$b(3.6,w_2) = 0.809$$

and

$$a(3.6,w_2) = -102.3 \text{ acre-ft}.$$

Therefore, the predictions for x = 3.6 mi are

$$Q(3.6,w_2) = -102.3 + 0.809(700) = 464.0 \text{ acre-ft}$$

for the volume and

$$q(3.6,w_2) = -238.0 + 0.809(4{,}000) = 2{,}998 \text{ cfs}$$

for the peak rate. For distances beyond this point, the flow will be contained in the channel banks. The parameters for in-bank flow with a distance of x = 10.0 − 3.6 = 6.4 mi are

$$a = -0.00465KD = -0.1674 \text{ acre-ft},$$

$$k = -1.09 \ln\left[1 - 0.00545\frac{KD}{P}\right] = 0.000461 \text{ (ft-mi)}^{-1},$$

and

$$b = e^{-k} = e^{-0.000461} = 0.99954$$

* *Average hydraulic conductivity from table 19–1.*

† *Includes width w_1.*

for K = 3.0, D = 12, and P = 464.0 acre-ft, which is the inflow from the upstream reach. With these unit channel parameters, the parameters for in-bank flow are

$$b(6.4,w_1) = e^{-kxw_1} = e^{-(0.000461)(6.4)(150)} = 0.642$$

and

$$\begin{aligned} a(6.4,w_1) &= \frac{a}{1-b}[1 - b(x,w_1)] \\ &= \frac{-0.1674}{(1 - 0.99954)}[1 - 0.642] \\ &= -130.3 \text{ acre-ft.} \end{aligned}$$

The predicted outflow is

$$\begin{aligned} Q(6.4,w_1) &= -130.3 + 0.642(464.0) \\ &= 167.6 \text{ acre-ft} \end{aligned}$$

for the volume and

$$\begin{aligned} q(6.4,w_1) &= -298.9 + 0.642(2{,}998) \\ &= 1{,}626 \text{ cfs} \end{aligned}$$

for the peak discharge. Therefore, the prediction is out-of-bank flow for about 3.6 mi and in-bank flow for 6.4 mi, with an outflow volume of 168 acre-ft and a peak discharge of 1,626 cfs.

This example illustrates the need for judgment in applying the procedure for estimating losses in out-of-bank flow. Care must be taken to ensure that transmission losses do not reduce the flow volume and peak to the point where flow is entirely within the channel banks. If this occurs, then the reach length must be broken into subreaches, as illustrated in this example.

Example 4. Transmission Losses Limited by Available Storage

In some circumstances, an alluvial channel could be underlain by nearly impervious material that might limit the potential storage volume in the alluvium (V) and thereby limit the potential transmission losses. Once the transmission losses fill the available storage, nearly all additional inflow will become outflow; the procedure is modified to predict and apply this secondary threshold volume, P_1.

Given: The channel reach in Example 1 with total available storage (maximum potential transmission loss) of V = 30 acre-ft. Given the volume equation from Case 1 of Example 1, compute equations to apply after the potential losses are satisfied. From Example 1, $a(x,w) = -10.38$ acre-ft, $b(x,w) = 0.850$, and $P_o(x,w) = 12.21$ acre-ft.

Solution: The total losses are $P - Q(x,w)$ computed as

$$\begin{aligned} P - [a(x,w) + b(x,w)P] = &-a(x,w) \\ &+ [1 - b(x,w)]P. \end{aligned}$$

Equating this computed loss to V and solving for the inflow volume predicts the inflow volume above which only the maximum alluvial storage is subtracted,

$$P_1 = \frac{V + a(x,w)}{1 - b(x,w)}.$$

For this example, this threshold inflow volume is 130.8 acre-ft. With this additional threshold, the prediction equation for outflow volume is modified to

$$Q(x,w) = \begin{cases} 0 & P \leq P_o(x,w) \\ a(x,w) + b(x,w)P & P_o(x,w) \leq P \leq P_1 \\ P - V & P > P_1. \end{cases} \tag{19–18}$$

For the example being discussed, the solution to this general equation is

$$Q(x,w) = \begin{cases} 0 & P \leq 12.21 \\ -10.38 + 0.850P & 12.21 \leq P \leq 130.8 \\ P - 30 & P > 130.8 \end{cases}$$

The slope of the regression line is equal to $Q(x,w)/[P - P_o(x,w)]$, so an equivalent slope, once the available storage is filled, is $b_{eq} = (P - V)/[P - P_o(x,w)]$, which for this example is $b_{eq} = (P - 30)/(P - 12.21)$. For an inflow volume of P = 300 acre-ft and p = 3,000, the equivalent slope is $b_{eq} = 0.938$. Using the equivalent slope, the peak equation is

$$\begin{aligned} q(x,w) &= \frac{-12.1}{D}[P - Q(x,w)] + b_{eq}P \\ &= -90.75 + 0.938(3{,}000) = 2{,}723 \text{ cfs.} \end{aligned}$$

Therefore, the predicted outflow is Q(x,w) = 270 acre-ft and q(x,w) = 2,723 cfs.

If the storage limitation had been ignored, the original equations would have predicted an outflow volume of 245 acre-ft and a peak rate of outflow of 2,384 cfs. If a channel reach has limited available storage, the procedure should be modified, as it was in Example 4, to compute losses that do not exceed the available storage.

Summary

The examples presented illustrate the wide range of applications of the transmission loss procedures described in this chapter. The examples were chosen to emphasize some limitations and the need for sound engineering judgment. These concepts are summarized in table 19–4.

Table 19–4.—Outline of examples and comments on their applications

Example	Procedure	Special circumstances	Comments
1 (Case 1)	Table 19–2	Observed data available	Slope and intercept must satisfy the constraints
1 (Case 2)	Table 19–3	No observed data	Typical application
2	Table 19–3 Eqs. 19–4, 19–5	Uniform lateral inflow	Importance of lateral inflow demonstrated
3	Table 19–3 Eq. 19–17	Out-of-bank flow	Judgment required to interpret results
4	Table 19–2 Eq. 19–18	Limited available storage	Concept of equivalent slope used

Appendices

These appendices provide the reference material, derivations, and analyses of available data upon which the material presented in Chapter 19 is based. The basic procedure is outlined, and sources for additional information are provided.

Appendix 1—Derivation of Procedures for Estimating Transmission Losses When Observed Data Are Available

In much of the Southwestern United States, watersheds are characterized as semiarid with broad alluvium-filled channels that abstract large quantities of streamflow (Babcock and Cushing 1941; Burkham 1970a, 1970b; Renard 1970). These abstractions or transmission losses are important because streamflow is lost as the flood wave travels downstream, and thus runoff volumes are reduced. Although these abstractions are referred to as losses, they are an important part of the water balance. They diminish streamflow, support riparian vegetation, and recharge local aquifers and regional ground water (Renard 1970).

Simplified procedures have been developed to estimate transmission losses in ephemeral streams. These procedures include simple regression equations to estimate outflow volumes (Lane, Diskin, and Renard 1971) and simplified differential equations for loss rate as a function of channel length (Jordan 1977). Other, more complicated methods have also been used (Lane 1972, Wu 1972, Smith 1972, Peebles 1975).

Lane, Ferreira, and Shirley (1980) developed a procedure to relate parameters of the linear regression equations (Lane, Diskin, and Renard 1971) to a differential equation coefficient and the decay factor proposed by Jordan (1977). This linkage between the regression and differential equations provides the basis of the applications described in this chapter.

Empirical Basis of the Regression Equation

When observed inflow-outflow data for a channel reach of an ephemeral stream with no lateral inflow are plotted on rectangular coordinate paper, the result is often no outflow for small inflow events, with outflow increasing as inflow increases. When data are fitted with a straight-line relationship, the intercept on the X axis represents an initial abstraction. Graphs of this type suggest equations of the form

$$Q(x,w) = \begin{cases} 0 & P \leq P_o(x,w) \\ a(x,w) + b(x,w)P & P > P_o(x,w). \end{cases} \tag{19–1}$$

By setting $Q(x,w) = 0.0$ and solving for P, the threshold volume, the volume of losses that occur before outflow begins, is

$$P_o(x,w) = \frac{-a(x,w)}{b(x,w)}. \tag{19–2}$$

Differential Equation for Changes in Volume: Linkage With the Regression Model

Differential equations can be used to approximate the influence of transmission losses on runoff volumes. Because the solutions to these equations can be expressed in the same form as the regression equations, least-squares analysis can be used to estimate parameters in the transmission loss equations.

Unit Channel

The rate of change in volume, Q (as a function of arbitrary distance), with changing inflow volume, P, can be approximated as

$$\frac{dQ}{dx} = -c - k\,Q(x). \tag{19–19}$$

Substituting the initial condition and defining $P = Q(x = 0)$, the solution of equation 19–19 is

$$Q(x) = -\frac{c}{k}(1 - e^{-kx}) + Pe^{-kx}. \tag{19–20}$$

For a unit channel, equation 19–20 becomes

$$Q = -\frac{c}{k}(1 - e^{-k}) + Pe^{-k}, \tag{19–21}$$

which corresponds to the regression equation

$$Q = a + bP. \tag{19–22}$$

Equating equations 19–21 and 19–22, it follows that

$$b = e^{-k} \tag{19–11}$$

and

$$a = -\frac{c}{k}(1 - e^{-k}) = -\frac{c}{k}(1 - b) \tag{19–23}$$

are the linkage equations. Equation 19–23 can be solved for c as

$$c = -k\frac{a}{1 - b}.$$

Channel of Arbitrary Length and Width

For a channel of width w and length x,

$$\frac{dQ}{dx} = -wc - wkQ(x,w),$$

where $c = -k\frac{a}{1 - b}$, so that the differential equation is

$$\frac{dQ}{dx} = wk\frac{a}{1 - b} - wkQ(x,w).$$

Defining P as $Q(x = 0)$ and substituting this initial condition, the solution is

$$Q(x,w) = \frac{a}{1 - b}[1 - e^{-kxw}] + Pe^{-kxw}.$$

From the linkage

$$b(x,w) = e^{-kxw} \tag{19–13}$$

and

$$\begin{aligned} a(x,w) &= \frac{a}{1 - b}[1 - b(x,w)] \\ &= \frac{a}{1 - b}[1 - e^{-kxw}], \end{aligned} \tag{19–14}$$

where a and b are unit channel parameters and k is the decay factor.

Influence of Uniform Lateral Inflow

If Q_L is the uniform lateral inflow (acre-feet per

mile), this inflow becomes an additional term in the differential equation

$$\frac{dQ}{dx} = wk\frac{a}{1 - b} - wkQ(x,w) + Q_L.$$

The solution is

$$Q(x,w) = \frac{a}{1 - b}[1 - e^{-kxw}] + Pe^{-kxw} + \frac{Q_L}{kw}[1 - e^{-kxw}],$$

and through the linkage, the outflow volume equation for upstream inflow augmented by uniform lateral inflow is

$$Q(x,w) = a(x,w) + b(x,w)P + \frac{Q_L}{kw}[1 - b(x,w)]. \qquad (19\text{–}4)$$

Approximations for Peak Discharge

The basic assumption for peak discharge, q(x,w), is that the outflow peak, once an average loss rate has been subtracted, is equal to b(x,w) times the peak of the inflow hydrographs, p. That is, assume that

$$q(x,w) = -\frac{P - Q(x,w)}{D} + b(x,w)p,$$

where $P - Q(x,w) = -a(x,w) + [1 - b(x,w)]P$, so that

$$q(x,w) = \frac{12.1}{D}(a(x,w) - [1 - b(x,w)]P) + b(x,w)p, \qquad (19\text{–}3)$$

where D is the mean duration of flow and 12.1 converts acre-feet per hour to cubic feet per second. For a peak lateral inflow rate of q_L (cfs/ft), uniform along the reach, the peak discharge equation becomes

$$q(x,w) = \frac{12.1}{D}(a(x,w) - [1 - b(x,w)]P) + b(x,w)p + \frac{q_L(5{,}280)}{kw}[1 - b(x,w),$$

where 5,280 converts cubic feet per second per foot to cubic feet per second per mile.

For small inflows, where the volume of transmission losses is about equal to the volume of inflow, the peak discharge equation, equation 19–3, overestimates the peak rate of outflow. The relation between peak rate of outflow observed and that computed from equation 19–3 is shown in figure 19–1. The bias shown in figure 19–1 is for small events and tends to overpredict, but the equation does well for the larger events. The computed values shown in figure 19–1 were based on the mean duration of flow for each channel reach. Better agreement of predicted and observed peak rates of outflow might be obtained by using actual flow durations.

Appendix 2—Analysis of Selected Data Used to Develop the Procedure for Estimating Transmission Losses

So that parameters of the prediction equations could be related to hydrograph characteristics and to effective hydraulic conductivity, it was necessary to analyze selected data. Events involving little or no lateral inflow were selected from channel reaches in Arizona, Kansas, Nebraska, and Texas (table 19–5).

The data shown in table 19–5 are not entirely consistent because the events were floods of different magnitudes. The Walnut Gulch data are from a series of small to moderate events representing in-bank flow, whereas the Queen Creek data are for relatively larger floods and no doubt include some out-of-bank flow. The Trinity River data represent pumping diversions entirely within the channel banks. Data for the Kansas-Nebraska streams represent floods of unknown size, and may include out-of-bank flow.

The data summarized in table 19–5 were subjected to linear regression analysis to estimate the parameters a(x,w), b(x,w), $P_o(x,w)$, and kxw. These parameters are summarized in table 19–6. Parameters for the unit channels were computed for 10 channel reaches and are shown in table 19–7.

Appendix 3.—Estimating Transmission Losses When No Observed Data Are Available

Estimating transmission losses when observed inflow-outflow data are not available requires a technique for using effective hydraulic conductivity to develop parameters for the regression analysis.

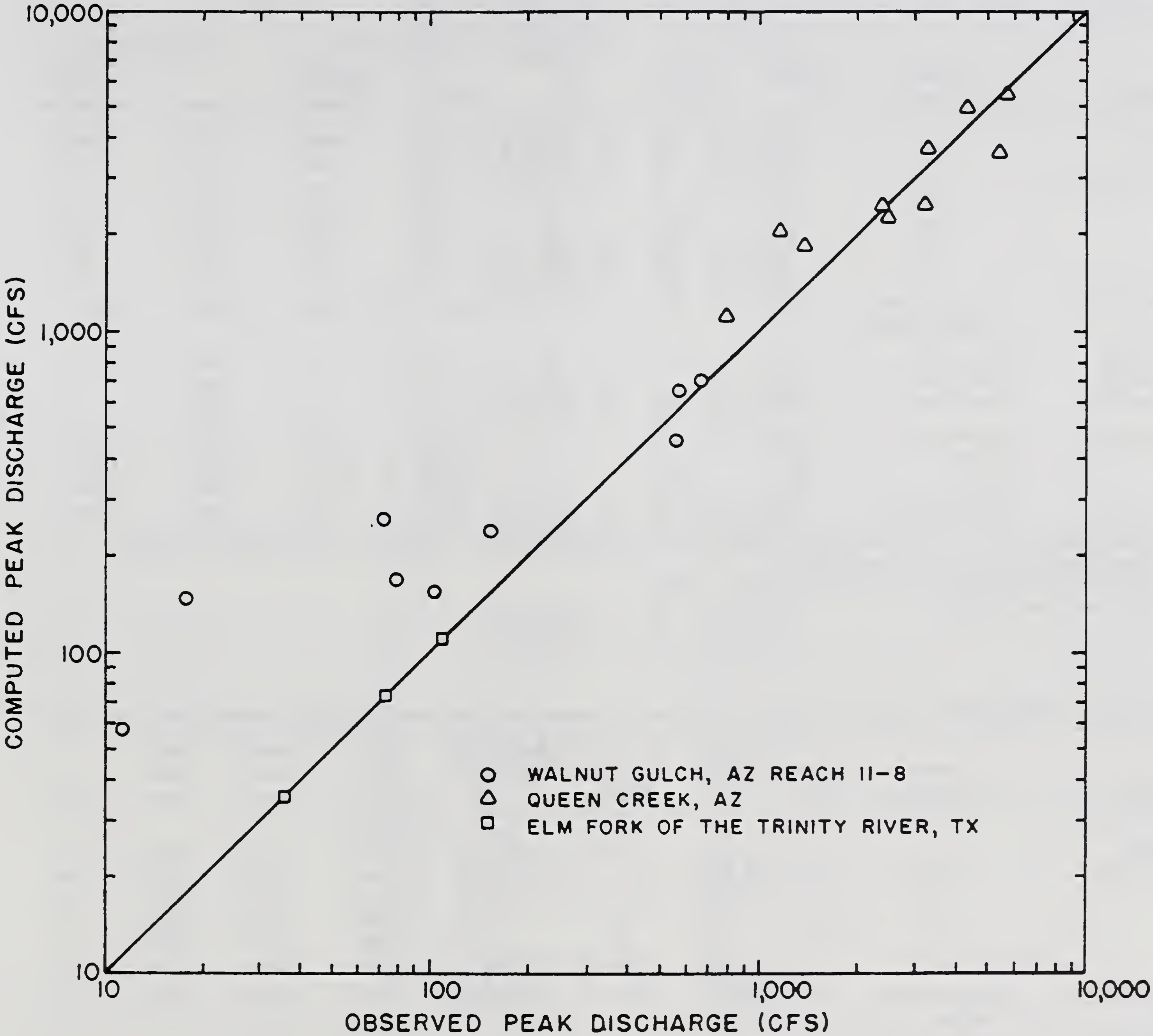

Figure 19–1.—Observed vs. computed peak discharge of the outflow hydrograph.

Table 19–5.—Hydrologic data used in analyzing transmission losses (Lane et al. 1980)

Location	Reach identification	Length, x	Average width, w	Number of events	Inflow volume		Outflow volume	
					Mean	Standard deviation	Mean	Standard deviation
		mi	*ft*		*acre-ft*	*acre-ft*	*acre-ft*	*acre-ft*
Walnut Gulch, Ariz.[1]	11–8	4.1	38	11	16.5	14.4	8.7	11.4
	8–6	0.9	—	3	13.7	—	11.4	—
	8–1	7.8	—	3	16.3	—	1.6	—
	6–2	2.7	107	30	75.1	121.6	59.9	101.0
	6–1	6.9	121	19	48.3	51.7	17.1	26.5
	2–1	4.2	132	32	49.3	42.7	24.4	31.4
Queen Creek, Ariz.[2]	Upper to lower gaging station	20.0	277	10	4,283	5,150	2,658	3,368
Elm Fork of Trinity River, Tex.[3]	Elm Fork–1	9.6	—	3	454	—	441	—
	Elm Fork–2	21.3	—	3	441	—	424	—
	Elm Fork–3	30.9	120	3	454	—	424	—
Kansas-Neb.[4]	Prairie Dog	26.0	17	5	1,890	1,325	1,340	1,218
	Beaver	39.0	14	7	2,201	2,187	1,265	1,422
	Sappa	35.0	23	6	6,189	8,897	3,851	7,144
	Smokey Hills	47.0	72	4	1,217	663	648	451

[1] Data on file at USDA-ARS, Southwest Rangeland Water Research Center, 442 E. 7th Street, Tucson, AZ 85705.
[2] Data from Babcock and Cushing (1941).
[3] Data from the Texas Board of Water Engineers (1960).
[4] Data from Jordan (1977).

Table 19–6.—Parameters for regression model and differential equation model for selected channel reaches (Lane et al. 1980)

Location	Reach identification	Reach no.	Length, x	Average width, w	Regression intercept, a(x,w)	Model slope, b(x,w)	Threshold volume, $P_o(x,w)$	Decay factor, kxw	R^2
			mi	*ft*	*acre-ft*	*acre-ft*			
Walnut Gulch, Ariz.	11–8	1	4.1	38	−4.27	0.789	5.41	0.2370	0.98
	8–6	2	0.9	—	−0.34	0.860	0.40	0.1508	.99
	8–1	3	7.8	—	−2.38	0.245	9.71	1.4065	.84
	6–2	4	2.7	107	−4.92	0.823	5.98	0.1948	.98
	6–1	5	6.9	121	−5.56	0.469	11.86	0.7572	.84
	2–1	6	4.2	132	−8.77	0.673	13.03	0.3960	.84
Queen Creek, Ariz.	Upper to lower station	7	20.0	277	−117.2	0.648	180.90	0.4339	.98
Elm Fork of Trinity River, Tex.	Elm Fork–1	8	9.6	—	−15.0	[1]1.004	—	—	.99
	Elm Fork–2	9	21.3	—	[1]+7.6	0.944	—	—	.99
	Elm Fork–3	10	30.9	120	−8.7	0.952	9.14	0.0492	.99
Kansas-Nebraska	Prairie Dog	11	26.0	17	−353.1	0.896	394.10	0.1098	.95
	Beaver	12	39.0	14	−157.3	0.646	243.50	0.4370	.99
	Sappa	13	35.0	23	−1,076.3	0.796	1,352.10	0.2282	.98
	Smokey Hills	14	47.0	72	−99.1	0.614	161.40	0.4878	.81

[1] Channel reaches where derived regression parameters did not satisfy the constraints.

Table 19–7.—Unit length, unit width, and unit length and width parameters for selected channel reaches (Lane et al. 1980)

Location	Identification	Unit length parameters			Unit width parameters			Unit length and width parameters			
		a(w)	b(w)	P_o(w)	a(x)	b(x)	P_o(x)	a	b	P_o	k
Walnut Gulch, Ariz.	11–8	−1.13657	0.94384	1.2042	−0.12587	0.99378	0.1267	−0.03076	0.998480	0.0308	0.001521
	6–2	−1.93484	0.93039	2.0796	−0.05059	0.99818	0.0507	−0.01874	0.999326	0.0187	0.000674
	6–1	−1.08819	0.89607	1.2144	−0.06541	0.99376	0.0658	−0.00950	0.999094	0.0095	0.000907
	2–1	−2.41320	0.91002	2.6518	−0.08046	0.99700	0.0807	−0.01915	0.999286	0.0192	0.000714
Queen Creek, Ariz.	Upper to lower station	−7.14508	0.97854	7.3018	−0.52273	0.99843	0.5236	−0.02597	0.999922	0.0260	0.0000783
Trinity River, Tex.	Elm Fork–3	−0.28825	0.99841	0.2887	−0.07427	0.99959	0.0743	−0.002404	0.999987	0.0024	0.0000133
Kansas-Nebraska	Prairie Dog	−14.30986	0.99579	14.3705	−21.86124	0.99356	22.0029	−0.842008	0.999752	0.8422	0.000248
	Beaver	−4.95071	0.98886	5.0065	−13.65447	0.96927	14.0874	−0.355480	0.999200	0.3558	0.000800
	Sappa	−34.28091	0.99350	34.5052	−52.07808	0.99013	52.5972	−1.493102	0.999717	1.4935	0.000283
	Smokey Hills	−2.65060	0.98968	2.6782	−1.73337	0.99325	1.7451	−0.036970	0.999856	0.0370	0.000144

Estimating Effective Hydraulic Conductivity

The total volume of losses for a channel reach is KD, where K is the effective hydraulic conductivity and D is the duration of flow. Also, the total losses are P − Q(x,w), so that

$$KD = 0.0275[P - Q(x,w)],$$

where 0.0275 converts acre-feet per foot-mile-hour to inches per hour. Or, solving for K,

$$K = \frac{0.0275\,[P - Q(x,w)]}{D}.$$

But

$$P - Q(x,w) = -a(x,w) + [1 - b(x,w)]P,$$

so that

$$K = \frac{0.0275}{D}[-a(x,w) + [1 - b(x,w)]P] \qquad \textbf{(19–24)}$$

is an expression for effective hydraulic conductivity. If mean values for D and P are used, then equation 19–24 estimates the mean value of the effective hydraulic conductivity.

Effective Hydraulic Conductivity vs. Model Parameters

For a unit channel, outflow is the difference between inflow and transmission losses:

$$Q = P - KD.$$

Because $Q = a + bP$,

$$-a + (1 - b)P = KD.$$

But because a and (1 − b)P are in acre-feet and KD, the product of conductivity and duration, is in inches, the dimensionally correct equation is

$$-a + (1 - b)P = 0.0101KD,$$

where 0.0101 converts inches over a unit channel to acre-feet. Because this equation is in two unknowns (a and b), an additional relationship is required to solve it. As a first approximation, the total losses are partitioned between the two terms in the equation. That is, let

$$a = -\alpha(0.0101KD)$$

and

$$(1 - b) = (1 - \alpha)\left(0.0101\frac{KD}{P}\right).$$

Solving for b,

$$b = 1 - (1 - \alpha)\left(0.0101\frac{KD}{P}\right),$$

where $0 \leq \alpha \leq 1$ is a weighting factor. Solve for k by substituting $b = e^{-k}$ and taking the negative natural log of both sides, i.e.,

$$k = -\ln\left[1 - (1 - \alpha)\left(0.0101\frac{KD}{P}\right)\right].$$

The selected data were analyzed to determine α by least-squares fitting as shown in table 19–8. For the data shown in table 19–8, the estimate of α was 0.46. Figures 19–2 and 19–3 show the data in table 19–8 plotted according to the equations

$$a = -0.00465KD \qquad \textbf{(19–15)}$$

and

$$k = -1.09\ln\left[1 - 0.00545\frac{KD}{P}\right], \qquad \textbf{(19–16)}$$

where for each channel reach, mean values were used for K, D, and P. These relationships were used to calculate the values shown in table 19–1.

Auxiliary data compiled in a report by Wilson et al. (1980) are shown in table 19–9. Although the estimates of infiltration rates were obtained by a variety of methods, most rates were based on streamflow data. Because these estimates generally involved longer periods of flow than in the smaller ephemeral streams, they should be representative of what is called effective hydraulic conductivity. The data show the range of estimates of hydraulic conductivity for various streams within a river basin as estimated by several investigators. For this reason, they should be

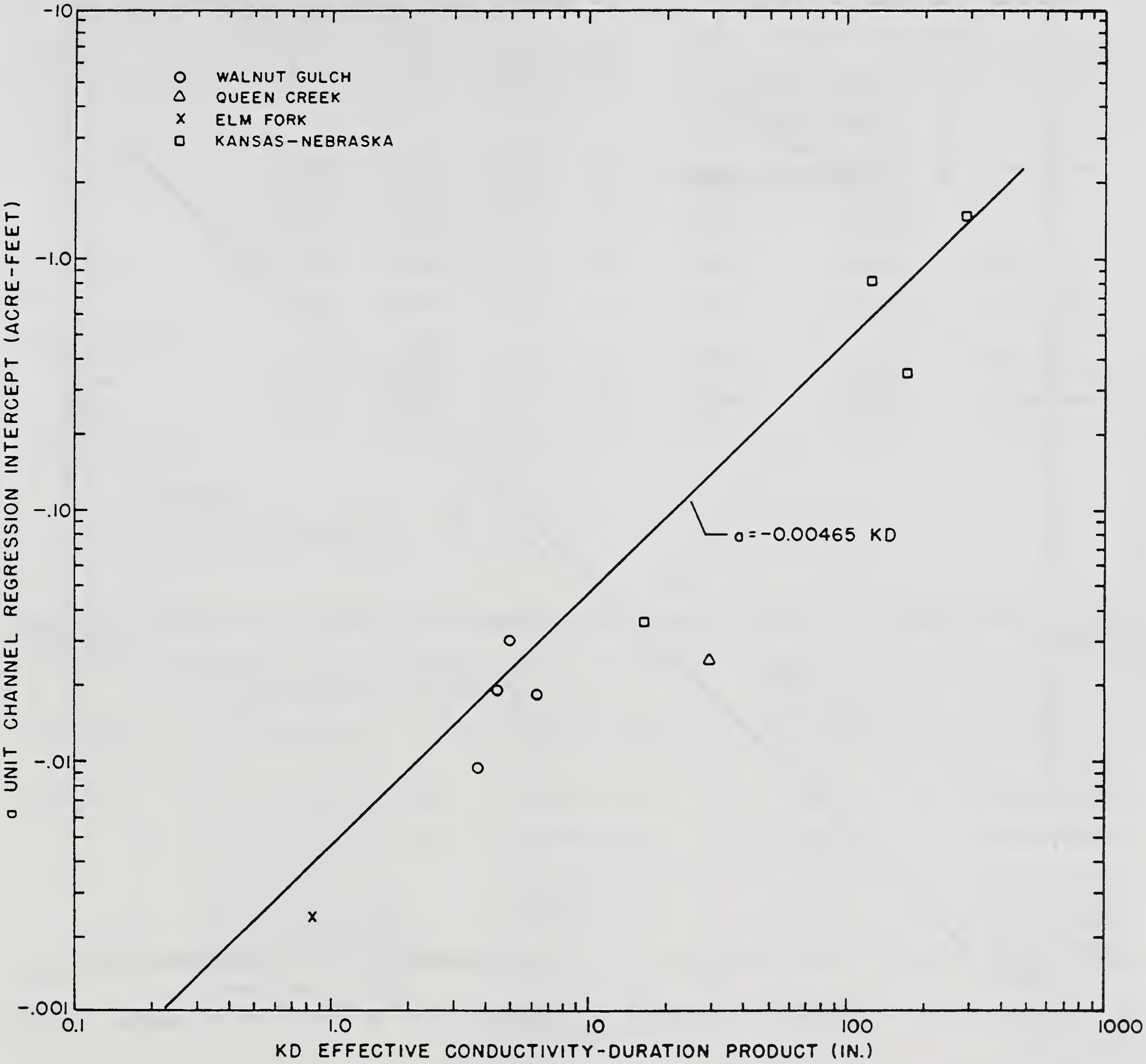

Figure 19-2.—Relation between KD and regression intercept.

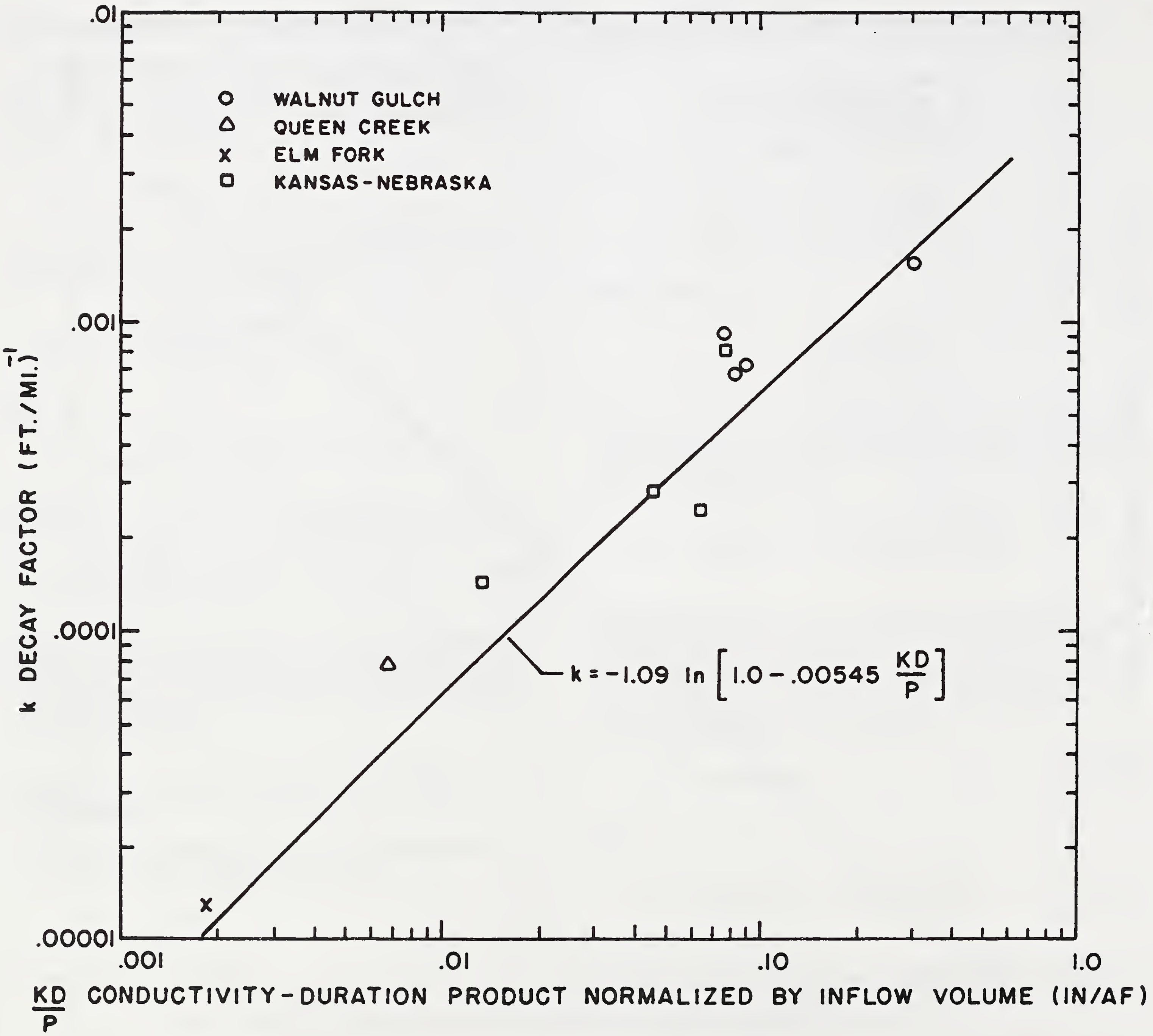

Figure 19-3.—Relation between KD/P and decay factor.

Table 19–8.—Data for analysis of relations between effective hydraulic conductivity and model parameters (Lane et al. 1980)

Location	Unit channel intercept, a	Decay factor, k	K	KD	$\frac{KD}{P}$	$-\ln\left[1 - 0.00545\frac{KD}{P}\right]$	Comments
	acre-ft	*(ft-mi)$^{-1}$*	*in/hr*	*in*	*in/acre-ft*		
Walnut Gulch							
11–8	−0.03076	0.001521	1.55	4.96	0.3010	0.001643	In-bank flow
6–2	−0.01874	0.000674	1.36	6.26	0.0834	0.000455	
6–1	−0.00950	0.000907	1.03	3.71	0.0768	0.000419	
2–1	−0.01915	0.000714	1.11	4.44	0.0901	0.000492	
Queen Creek	−0.02597	0.0000783	0.54	29.16	0.0068	0.0000371	Mixed flow
Elm Fork	−0.00240	0.0000133	0.01	0.84	0.0019	0.0000104	In-bank flow
Kansas-Nebraska							
Prairie Dog	−0.84201	0.000248	1.28	122.9	0.0650	0.000355	Mixed flow:
Beaver	−0.35548	0.000800	1.38	169.7	0.0771	0.000421	average widths
Sappa	−1.49310	0.000283	2.57	287.8	0.0465	0.000254	may be under-
Smokey Hills	−0.03697	0.000144	0.17	16.3	0.0134	0.000073	estimated

Least-squares fit:

$$a = -0.00465KD$$

$$k = -1.09 \ln\left[1 - 0.000545\frac{KD}{P}\right]$$

Table 19–9.—Auxiliary transmission-loss data for selected ephemeral streams in southern Arizona (data taken from Wilson et al. [1980])

River basin	Stream reach	Estimation method	Effective hydraulic conductivity	Source of estimates
			in/hr	
Santa Cruz	Santa Cruz River, Tucson to Continental	Streamflow data[1]	1.5–3.4	Matlock (1965)
	Santa Cruz River, Tucson to Cortero	Streamflow data	3.2–3.7	Matlock (1965)
	Rillito Creek, Tucson	Streamflow data	0.5–3.3	Matlock (1965)
	Rillito Creek, Cortero	Streamflow data	2.2–5.5	Matlock (1965)
	Pantano Wash, Tucson	Streamflow data	1.6–2.0	Matlock (1965)
	Average for Tucson area	—	1.65	Matlock (1965)
Gila	Queen Creek	Streamflow data:		Babcock and Cushing (1942)
		Summer flows	0.07–0.52	
		Winter flows	0.37–1.05	Babcock and Cushing (1942)
		Average for all events	0.54	Babcock and Cushing (1942)
		Seepage losses in pools[2]	>2.0	Babcock and Cushing (1942)
	Salt River, Granite Reef Dam to 7th Avenue	Streamflow data	0.75–1.25	Briggs and Werho (1966)
San Pedro	Walnut Gulch	Streamflow data	1.1–4.5	Keppel (1960), Keppel and Renard (1962)
	Walnut Gulch	Streamflow data	2.4	Peebles (1975)
San Simon	San Simon Creek	—	0.18	Peterson (1962)

[1] Transmission losses estimated from streamflow data.
[2] Measurement of loss rates from seepage in isolated pools.

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HYDROLOGY

CHAPTER 20. WATERSHED YIELD

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1956

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CHAPTER 20. WATERSHED YIELD

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CHAPTER 20. WATERSHED YIELD

The water yield of a watershed, by years or seasons or months, is used in the planning and design of some watershed projects, especially those involving irrigation. The hydrologist supplies estimates of these yields, as required, or supplies methods adapted to specific local conditions by which others may make the estimates. This chapter contains general methods for estimating water yields on ungaged watersheds, with suggestions for such modifications as local conditions may justify.

Summary of Problems

Watershed yield is dependent on many physical factors, most of which usually cannot be quantitatively determined during ordinary field operations. Methods of estimating yield from ungaged watersheds may be classified as follows:

(a) Using only climatic factors. Examples are graphs or equations using precipitation and temperature, or only precipitation.

(b) Using only geographic location. Examples are maps having lines of equal runoff, or the practice of estimating yield by interpolation between gaged watersheds.

(c) Using watershed and climatic factors. Examples are (1) water accounting method, (2) regional analysis, and (3) use of figure 10-1 and daily rainfall.

The choice of method often rests on the type of runoff to be estimated, which may be classified as:

(a) Yield as a residual of precipitation after evapotranspiration. Examples are watersheds where base flow predominates. Water accounting methods are useful with this type.

(b) Yield as an excess of surface supply over watershed surface intake. Examples are watersheds where surface runoff predominates. Methods using rainfall and infiltration are needed, such as a method utilizing figure 10-1.

(c) Yield as a diverted flow. Examples are watersheds having irrigation projects that get their supply outside of the watershed and their return flows occur inside; or watersheds with surface runoff predominating, whose streams carry return or waste flows from irrigation projects or municipal and industrial plants that pump their supplies from deep wells or receive them from outside the watershed.

Instrumentation and watershed conditions may suggest or govern the choice of method. These conditions may vary with watershed size--that is, instrumentation or methods suitable for a small watershed having surface runoff may be unsuitable for a large watershed (into which the small one drains) that has a high percent of base flow. The conditions may similarly vary with geographic location, the presence of water tables, elevation, aspect, and latitude. Other factors that have influence can also be listed. However, evaluation of the listed and unlisted factors is still more properly a research activity. In practice, the primary factors that can ordinarily be considered for ungaged streams are: (1) streamflow on nearby watersheds, (2) precipitation, (3) hydrologic soil-cover complexes, (4) evapotranspiration, (5) temperature, (6) transmission losses, and (7) base flow accretions.

Determinations of water yield will usually have two types of error, (1) that due to insufficient recognition of the natural fluctuations of yield from year to year, and (2) that due to insufficient recognition of the most important influences on yield in a given watershed. The first type of error can be reduced by working with long records, the second by further studies of all possible major influences. However, increasing the time spent on yield estimates does not always assure greater accuracy in the estimates. Therefore, the methods given below should be considered as giving estimates so broad that the influence of specific factors have large margins of error.

Methods for Estimating Yields

A fuller account of such methods will be given in the National Engineering Handbook, Section 4, Hydrology.

Regional analysis

The general procedure is described in Section 2.8 of the Guide. For water yield, the method is used with annual, seasonal, or monthly flows of gaged watersheds. The slopes of the frequency lines will vary, being flattest for annual yields and becoming steeper (larger R on figure 18-3) as smaller divisions of a year are used.

This method is suitable for estimating the first two types of runoff mentioned above. It is readily adapted to watershed conditions, when data are available, since the watersheds can be selected for whatever factors can be used. However, the factors (and not the regional analysis method) may very strongly govern the accuracy of the results

for watershed yield. For example, if one of the important factors on the problem watershed is aspect, and it is too vaguely represented by the gaged watersheds used in the analysis, then the accuracy of the results of the regional analysis will suffer. Transmission losses, for example, may be insufficiently detected by this method, and additional field studies may be required to determine those losses.

Water accounting

This method is suitable for estimating the first type of runoff mentioned above. As presented here, the method is A. L. Sharp's modification and enlargement of a method proposed by C. W. Thornthwaite in Trans. Amer. Geophys. Union, pp. 686-693, April 1944. The transmission loss is not estimated by this method and must be determined by other methods (Chapter 19).

The flow chart in Chapter 10 will assist in understanding the following steps.

1. Obtain soils and land treatment data for the watershed.

2. Obtain estimates of the water-holding capacity of each soil or soil group, expressed as inches depth of water between the amounts at field capacity and wilting point. The soil depth for which this capacity is needed is the depth of the intensive root zone, or 3 feet, whichever is lesser.

3. Compute the water-holding capacity of the watershed, weighting by areal extent of the soils or soil groups.

4. Obtain watershed cover data for the season or seasons for which yields are to be estimated. Data needed are (1) types of cover, and (2) areal extent.

5. Compute potential evapotranspiration (potential ET), or consumptive use by months for each major crop or land use. The Blaney-Criddle method of computing potential ET is generally used as given in "Determining Water Requirements in Irrigated Areas from Climatological and Irrigation Data," by Harry F. Blaney and Wayne D. Criddle, Soil Conservation Service, U.S.D.A., SCS-TP-96, Washington, D. C., revised 1952.

6. Compute monthly weighted potential ET for the watershed.

7. Obtain monthly rainfall data for the watershed, for a period of years estimated to be long enough to give adequate yield values (see Chapter 18 on length of record). The estimate of length should be made after previous use of figure 18-3 with other yield data in the vicinity.

8. Compute average rainfall over the watershed, by months, for each year of record.

9. Tabulate rainfall and ET data as shown on table 20-1, and compute runoff, by months, for each year of record.

   (a) In table 20-1, the computation starts with a month when available soil moisture is <u>fully depleted</u>. It could start equally well with a month when the soils are <u>fully saturated</u>.

   (b) If there is a break in the year, as in table 20-1, the first month after the break should have either of the moisture conditions given in (a) above.

   (c) When the precipitation is snowfall, convert to water equivalent (watershed average) before using in line 1 (see Chapter 11 for methods). Watersheds consistently having snowfall on one portion and rainfall on the other should be subdivided and the yields of the subdivisions computed separately, then combined for total watershed yield.

   (d) Work with subdivisions if the watershed soils differ in water-holding capacities by more than about 100% of the smallest capacity or by more than about 1 inch, whichever is greater.

   (e) Work with subdivisions if the watershed precipitation consistently varies widely in amount at different localities. This may be determined using average annual precipitation. The variation over a watershed (or subdivision) should not be greater than about 30% of the smallest value, or about 3 inches, whichever is greater.

10. After completion of the computations for the selected length of record, test the runoff estimates for adequacy of length of record, using the method of Chapter 18. The test should be made with values that will be used in planning or design. For example, if annual values are to be used, when they are tested; if monthly values are to be used, then all October values are tested separately, next all November, and so on. If the length of record is not adequate, additional years of precipitation are added and the yield computations extended.

Transmission losses are subtracted after Step 10. If these losses are proportionately large, it may be necessary to test the modified yields for adequacy of length of record.

Table 20-1. Sample computation by water accounting method.

Line	Item	October	November	December	January	February	March	April	May	Seasonal runoff
	All units in inches				1947 - 1948					
1	1/ Average rainfall	5.65	1.04	1.88	2.41	2.34	5.48	10.04	1.34	
2	2/ Initial soil moisture	0.00 3/	2.87	1.74	2.62	3.20	3.20	3.20	3.20	
3	Total available moisture	5.65	3.91	3.62	5.03	5.54	8.68	13.24	4.54	
4	4/ Potential evapotranspiration	2.78	2.17	1.00	0.90	1.00	2.69	3.18	3.89	
5	5/ Actual evapotranspiration	2.78	2.17	1.00	0.90	1.00	2.69	3.18	3.89	
6	Remaining available moisture	2.87	1.74	2.62	4.13	4.54	5.99	10.06	0.65	
7	6/ Final soil moisture	2.87	1.74	2.62	3.20	3.20	3.20	3.20	0.65	
8	Runoff	0.00	0.00	0.00	0.93	1.34	2.79	6.86	0.00	11.92
					1948 - 1949					
1	1/ Average rainfall	0.75	0.84	3.53	1.24	2.22	7.34	0.03	0.46	
2	2/ Initial soil moisture	0.00 3/	0.00	0.00	2.53	2.87	3.20	3.20	0.05	
3	Total available moisture	0.75	0.84	3.53	3.77	5.09	10.54	3.23	0.51	
4	4/ Potential evapotranspiration	2.78	2.17	1.00	0.90	1.00	2.69	3.18	3.89	
5	5/ Actual evapotranspiration	0.75	0.84	1.00	0.90	1.00	2.69	3.18	0.51	
6	Remaining available moisture	0.00	0.00	2.53	2.87	4.09	7.85	0.05	0.00	
7	6/ Final soil moisture	0.00	0.00	2.53	2.87	3.20	3.20	0.05	0.00	
8	Runoff	0.00	0.00	0.00	0.00	0.89	4.65	0.00	0.00	5.54

1/ Average over the watershed for each month of record.
2/ At start of month. Same as "Final soil moisture" for previous month.
3/ See text, Step 9, notes (a) and (b).
4/ Average annual values for the month.
5/ Total available moisture, or potential ET, whichever is smaller.
6/ At end of month. Same as "Initial soil moisture" for next month. This is never larger than the water-holding capacity determined in Step 3 of the text--in this case, 3.20 inches.

Note: Data are for a West Coast area of the United States, where the June-September precipitation is negligible.

Direct runoff method
Daily rainfall values and figure 10-1 can be used to estimate yields when these are of the second type described. Generally it may be assumed that direct runoff is being estimated. The procedure consists of using the method of Chapter 10 with all rainfalls. Snowmelt runoff is estimated separately using the methods of Chapter 11.

Table 9-1, which is used to determine curve numbers on figure 10-1, gives average values for the year. In using this table for yield estimates it is usually necessary to go into more detail about the cover, so that the weighted hydrologic soil-cover complex number varies not only for antecedent moisture conditions but also for the variation in cover throughout a given year and from year to year.

The direct runoff method is usually very tedious, since all daily precipitation in a long period of record must be accounted for, day by day, using soil-cover complex numbers that vary from month to month or even more often. The laboriousness of the procedure, however, does not guarantee close accuracy in the yield estimate.

Major errors with this method will generally be in the determinations of soil-cover complexes (which will vary through the year) and in antecedent moisture conditions (which will vary not only with precipitation and temperature, but also with soil-cover complexes). This method is more suitable for small watersheds than for large ones, since the large watersheds will have some base flow, which may be a significant proportion of total yield. Estimates by this method generally will have such a margin of error that the effects of individual factors should not be given much significance.

Climatic and geographic factors
In areas where there is no abrupt change in precipitation, hydrologic soil-cover complexes, or geology, yield may be readily estimated using maps with lines of equal runoff. Generalized national maps, such as Plate 1 of U.S.G.S. Circular 52, should be used with great caution. The text of the Circular, page 9, states that "Figure 2 and plate 1 should not be used to estimate runoff from ungaged areas." More localized maps, however, such as those prepared by John H. Dorroh, Jr. for the Southwestern States, will be very useful, especially where the advice of the map's originator may be sought.

K. M. Kent has used a form of the "direct runoff method" described above to prepare typical yield frequency lines for selected soil-cover complex numbers, which are used with a state map giving precipitation indices. Given the soil-cover complex number, the yield for a given frequency is quickly estimated for any locality in that state.

Graphs and equations of precipitation and temperature, or precipitation alone, have been used in the past much more than they are today. Figure 2 of U.S.G.S. Circular 52 is an example (but see remark about Plate 1). Such graphs and equations should be used with great caution since so many factors are ignored.

Discussion

Since so many factors enter into the estimating of yields, and since both the relative importance and quantitative influences of some factors are nearly always unknown, estimates of yield should be conservative, according to the use they will have. The planners and designers who will use the yield estimates will be best able to state the direction and degree of conservativeness required. The hydrologist can obtain the conservativeness by the use of the methods given above, and those in Chapter 18, Frequency Methods.

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SECTION 4

HYDROLOGY

CHAPTER 21. DESIGN HYDROGRAPHS

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CHAPTER 21. DESIGN HYDROGRAPHS

Introduction

This chapter contains a systematic approach to the development of design hydrographs for use in proportioning earth dams and their spillways according to SCS criteria. Included are data or sources of data for design rainfall amount, duration, and distribution; methods of modifying design runoff to include effects of channel losses, quick return flow, or upstream releases; and methods for rapid construction of hydrographs.

The methodology presented in this chapter is suitable for the design of many types of water control structures, including channel works, but the emphasis is on hydrology for design of earth dams that provide temporary storage for flood prevention in addition to permanent storage for other uses. Its chief purpose is to contribute to safe design. Although the methods are based on data of actual storms and floods, they are not intended for reproducing hydrographs of actual floods; more suitable methods for actual floods are found in earlier chapters.

The remainder of this chapter is divided into two major parts. The first is concerned with hydrologic design for principal spillways, the second for emergency spillways. The examples in each part go only as far as the completion of hydrographs. Methods of routing hydrographs through spillways are given in chapter 17. Uses of hydrographs are illustrated in other SCS publications.

Principal Spillways

The SCS criteria require principal spillway capacity and the associated floodwater retarding storage to be such that project objectives are met and that the frequency of emergency spillway operation is within specified limits. The criteria are met by use of a Principal Spillway Hydrograph (PSH) or its mass curve (PSMC), which are developed as shown in this part of the chapter. Details of SCS hydrologic criteria are given first, then details of the PSH and PSMC development are given in examples.

Any one of four methods of runoff determination is suitable for the design of principal spillway capacity and retarding storage. They are (1) the runoff curve number procedure using rainfall data and the watershed's characteristics, (2) the use of runoff volume maps covering specific areas of the United States, (3) the regionalization and transposition of volume-duration-probability analyses made by the SCS Central Technical Unit, and (4) the use of local streamflow data with provision of sufficient documentation on the method and results. The latter two methods are not discussed in this chapter because they vary in procedure from case to case, due to conditions of local data, and standard procedures have not yet been established.

Runoff Curve Number Procedure

The runoff curve number procedure uses certain climatic data and the characteristics of a watershed to convert rainfall data to runoff volume. This procedure should be used for those areas of the country not covered by runoff volume and rate maps. (Exhibit 21.1 through 21.5.)

SOURCES OF RAINFALL DATA. Rainfall data for the determination of direct runoff may be obtained from maps in U.S. Weather Bureau technical papers:

For durations to 1 day.--

TP-40. 48 contiguous States.
TP-42. Puerto Rico and Virgin Islands.
TP-43. Hawaii.
TP-47. Alaska.

For durations from 2 to 10 days.--

TP-49. 48 contiguous States
TP-51. Hawaii.
TP-52. Alaska.
TP-53. Puerto Rico and Virgin Islands.

AREAL ADJUSTMENT OF RAINFALL AMOUNT. If the drainage area above a structure is not over 10 square miles, no adjustment in rainfall amount is made. If it is over 10 square miles, the area-point ratios of table 21.1 may be used to reduce the rainfall amount. The table applies to all geographical locations serviced by SCS. The ratios are based on the 1- and 10-day depth-area curves of figure 10, U.S. Weather Bureau TP-49, but are modified to give a ratio of 1 at 10 square miles.

RUNOFF CURVE NUMBERS. The runoff curve number (CN) for the drainage area above a structure is determined and runoff is estimated as described in chapters 7 through 10. The CN is for antecedent moisture condition II and it applies to the 1-day storm used in development of the PSH or PSMC. If the 100-year frequency 10-day duration point

Table 21.1.--Ratios for areal adjustment of rainfall amount

Area	Area/point ratio for 1 day	10 days	Area	Area/point ratio for 1 day	10 days
sq. mi.			sq. mi.		
10 or less	1.000	1.000	80	937	0.968
15	.978	.991	100	[illegible]32	.966
20	.969	.986	[illegible]	[illegible]28	.964
25	.964	.983	[illegible]	.925	.962
30	.960	.981	[illegible]	.922	.961
35	.957	[illegible]	180	.920	.960
40	.95[illegible]	[illegible]	200	.918	.959
50	[illegible]	[illegible]	250	.914	.957
60	[illegible]	.972	300	.911	.956
70	.9[illegible]	.970	400	.910	.955

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rainfall for the structure site is 6 or more inches, the CN for the 10-day storm is taken from table 21.2. If it is less than 6 inches, the CN for the 10-day storm is the same as that for the 1-day storm. The 10-day CN is used only with the total 10-day rainfall.

CLIMATIC INDEX. The climatic index used in this part of the chapter is:

$$Ci = \frac{100\ Pa}{(Ta)^2} \qquad (21.1)$$

where Ci = climatic index
Pa = average annual precipitation in inches
Ta = average annual temperature in degrees Fahrenheit

Precipitation and temperature data for U.S. Weather Bureau stations can be obtained from the following Weather Bureau publications:

Climatological Data. Issued annually and monthly for each State or a combination of States and for Puerto Rico and Virgin Islands. The annual issues contain annual and monthly data and averages or departures; monthly issues contain similar information for individual months.

Climatic Summary of the United States - Supplement for 1931-1952. Issued once for each State or a combination of States.

Climates of the States. Issued once for each State and for Puerto Rico and Virgin Islands.

Monthly Normals of Temperature, Precipitation, and Heating Degree Days. Issued once for each State or a combination of States. Also contains annual averages.

Table 21.2.--Ten-day runoff curve numbers*

Runoff curve numbers for:					
1 day	10 days	1 day	10 days	1 day	10 days
100	100	80	65	60	41
99	98	79	64	59	40
98	96	78	62	58	39
97	94	77	61	57	38
96	92	76	60	56	37
95	90	75	58	[illegible]	36
94	88	74	57	[illegible]	35
93	86	73	[illegible]	[illegible]	34
92	84	72	[illegible]	52	33
91	82	[illegible]	[illegible]	51	32
90	[illegible]	[illegible]	52	50	32
89	[illegible]	69	51	49	31
88	[illegible]	68	50	48	30
87	70	67	48	47	29
86	74	66	47	46	28
85	72	65	46	45	27
84	71	64	45	44	27
83	69	63	44	43	26
82	68	62	43	42	25
81	66	61	42	41	24

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* This table is used only if the 100-year frequency 10-day point rainfall is 6 or more inches. If it is less, the 10-day CN is the same as that for 1 day.

Climatic Maps for the National Atlas. Maps with a scale of one in ten million. A map for average annual precipitation is available but there is no map for average annual temperature.

SCS personnel may obtain these publications through their Regional Technical Service Center.

CHANNEL LOSSES. If the drainage area above a structure has a climatic index less than 1, then the direct runoff from rainfall may be decreased to account for channel losses of influent streams. Channel losses can be determined from local data but the losses must not be more than determined by use of table 21.3. When adequate local data are not available, table 21.3 is to be used. Example 21.1 gives the procedure for making the channel loss reduction of direct runoff.

Channel losses in areas where the climatic index is 1 or more will require special study; results must be approved by the Director, Engineering Division, before being used in final design hydrology.

QUICK RETURN FLOW. Quick return flow (QRF) is the rate of discharge that persists for some period beyond that for which the 10-day PSH is derived. It includes base flow and other flows that become a part of the flood hydrograph such as (1) rainfall that has infiltrated and reappeared soon afterwards as surface flow; (2) drainage from marshes and potholes; and (3) delayed drainage from snow banks. If the drainage area above a structure has a climatic index greater than 1, then QRF must be added to the hydrograph or mass curve of direct runoff from rainfall. QRF can be determined from local data but it must not be less than the steady rate determined by use of table 21.4. When adequate local data are not available, table 21.4 is to be used. Example 21.2 gives the procedure for adding QRF to the hydrograph or mass curve of direct runoff derived from rainfall.

UPSTREAM RELEASES. Releases from upstream structures must be added to the hydrograph or mass curve of runoff. This addition must be made regardless of other additions or subtractions of flow. Upstream release rates are determined from routings of applicable hydrographs or mass curves through the upstream structures and the reaches downstream from them.

COMBINATIONS OF CHANNEL LOSS, QUICK RETURN FLOW AND UPSTREAM RELEASE. In the introduction it was stated that the chief purpose of the methodology in this chapter is to contribute to safe design and that these methods are not intended for reproducing actual floods. Equation 21.1 and tables 21.1 through 21.4 must be considered in that light.

For large watersheds the topography may be such that two climatic indexes are needed, for example where a semiarid plain is surrounded by mountains. In such cases the design storm is determined for the watershed as a whole, the direct runoff is estimated separately for the two

TABLE 21.3--CHANNEL-LOSS FACTORS FOR REDUCTION OF DIRECT RUNOFF

DRAINAGE AREA	CLIMATIC INDEX CI						
	1.0	0.9	C.8	0.7	0.6	0.5	0.4 OR LESS
SQ. MI.							
1. OR LESS	1.00	1.00	1.00	1.00	1.00	1.00	1.00
2.	1.00	.98	.97	.95	.93	.90	.87
3.	1.00	.98	.95	.92	.89	.85	.80
4.	1.00	.97	.94	.90	.86	.81	.76
5.	1.00	.96	.92	.88	.84	.78	.73
6.	1.00	.96	.92	.87	.82	.76	.70
7.	1.00	.96	.91	.86	.81	.75	.68
8.	1.00	.95	.90	.85	.79	.73	.66
9.	1.00	.95	.90	.84	.78	.72	.65
10.	1.00	.95	.89	.84	.77	.71	.63
20.	1.00	.93	.86	.79	.72	.64	.55
30.	1.00	.93	.85	.77	.69	.60	.51
40.	1.00	.92	.84	.75	.66	.57	.48
50.	1.00	.91	.83	.74	.65	.55	.46
60.	1.00	.91	.82	.73	.63	.54	.44
70.	1.00	.91	.81	.72	.62	.53	.43
80.	1.00	.90	.81	.71	.62	.52	.42
90.	1.00	.90	.80	.71	.61	.51	.41
100.	1.00	.90	.80	.70	.60	.50	.40
150.	1.00	.89	.78	.68	.57	.47	.37
200.	1.00	.89	.77	.66	.56	.45	.35
250.	1.00	.88	.77	.65	.54	.44	.33
300.	1.00	.88	.76	.64	.53	.42	.32
350.	1.00	.87	.75	.64	.52	.41	.31
400.	1.00	.87	.75	.63	.51	.41	.30

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Table 21.4. Minimum quick return flow for PSH derived from rainfall.

Ci	QRF in./day	QRF csm	Ci	QRF in./day	QRF csm
1.00	0	0	1.50	0.234	6.29
1.02	.011	.30	1.52	.239	6.43
1.04	.022	.60	1.54	.244	6.56
1.06	.033	.90	1.56	.249	6.70
1.08	.045	1.20	1.58	.254	6.83
1.10	.056	1.50	1.60*	.259	6.96
1.12	.067	1.80	1.65	.270	7.26
1.14	.078	2.10	1.70	[illegible]	7.53
1.16	.089	2.40	1.75	[illegible]	7.80
1.18	.100	2.70	[illegible]	[illegible]	8.04
1.20	.112	3.00	[illegible]	.308	8.28
1.22	.123	[illegible]	[illegible]	.318	8.55
1.24	.13[illegible]	[illegible]	1.95	.326	8.76
1.26	[illegible]	[illegible]	2.00	.335	9.00
1.28	[illegible]	[illegible].11	2.05	.343	9.22
1.30	.163	4.38	2.10*	.351	9.44
1.32	.171	4.60	2.20	.367	9.87
1.34	.180	4.84	2.30	.382	10.27
1.36	.188	5.06	2.40	.396	10.65
1.38	.195	5.24	2.50	.410	11.02
1.40	.202	5.43	2.60	.423	11.37
1.42	.209	5.62	2.70	.436	11.72
1.44	.216	5.81	2.80	.449	12.07
1.46	.222	5.97	2.90	.461	12.40
1.48	.228	6.13	3.00**	.473	12.72

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* Change in tabulation interval.

** For Ci greater than 3, use:

$$QRF = 9\,(Ci - 1)^{0.5} \text{ for QRF in csm}$$

or

$$QRF = 0.335\,(Ci - 1)^{0.5} \text{ for QRF in inches per day.}$$

parts by use of appropriate CN and then combined, the channel loss reduction is based on the area of the semiarid plain and its climatic index, the hydrograph or mass curve of direct runoff is constructed, and QRF from the mountain area is added.

If there are upstream structures, their releases are always added regardless of the downstream climatic index or other considerations.

Runoff Volume Maps Procedure

The runoff volume and rate maps, exhibits 21.1 through 21.5, are provided for areas of the United States where measured runoff volumes vary significantly from those obtained from the curve number procedure for converting rainfall to runoff. The mapped areas are of two general types: (1) the areas where runoff from either snowmelt, dormant season rainfall, or a combination of the two produce greater runoff volumes than growing season rainfall and (2) the deep snowpack areas of high mountain elevations.

AREAS OF MAPPED RUNOFF VOLUME. The 100-year 10-day runoff volume maps, exhibits 21.1 and 21.4, represent regionalized values derived from gaged streamflow data and supplemented with climatological data and local observations. These values should be used for estimating floodwater detention storage within the map area where local streamflow data are not adequate.

Areal reduction should not be made on the 10-day runoff volumes shown in the maps. Since these amounts were derived from stream gage data, base flow and channel loss will be automatically included in the map values and in Table 21.10.

Quick return flow in this procedure is used as the rate of discharge expected to persist beyond the flood period described under the 10-day PSH. The rates of discharge, exhibit 21.3, were derived by averaging the accumulated depths of runoff between the 15th and 30th day on volume-duration-probability (VDP) accumulation graphs. They were obtained from the same VDP station data from which the 100-year 10-day runoff volumes in exhibit 21.1 were obtained.

When using the Runoff Volume Maps Procedure, the quick return flow rate, exhibit 21.3, is made an extension to the PSH before routing it through the reservoir, figure 21.1a.

DEEP SNOWPACK AREAS. Flood volume estimates from the deep snowpack areas may be calculated from local streamflow data or by regionalization and transposition of streamflow data.

A standard procedure for making a regional analysis of volumes of runoff for varying durations and frequencies has not been developed at this time. Past experience has indicated that acceptable estimates can be made using multiple regression techniques. If watersheds can be selected that are reasonably homogeneous with regard to items

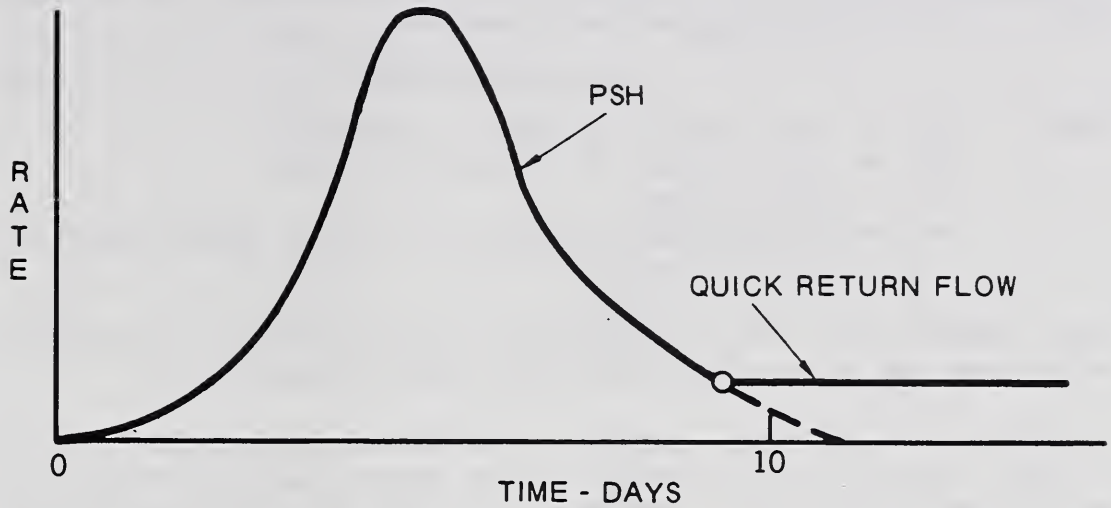


Figure 21.1a Quick Return Flow Combined with Principal Spillway Hydrograph for the Runoff Volume Maps Procedure.

such as seasonal precipitation, range of elevation, aspect, cover, geology, soils, etc., estimating equations can be developed with a minimum number of independent variables. Until techniques are developed to properly analyze the effects of a number of variables, the selection of homogeneous gaged watersheds with as much similarity to the ungaged watersheds as possible is recommended for estimating volume-duration-probability data. Statistics from volume-duration-probability studies of gaged watersheds can also be used to assist in developing estimating equations.

Construction of Principal Spillway Hydrographs and Mass Curves

The principal spillway capacity and retarding storage amount are proportioned using the Principal Spillway Hydrograph (PSH) or its mass curve (PSMC) developed from tabulations given in table 21.10. Examples in this section show how to select the appropriate set of tabulations and to construct the PSH or PSMC. One or more routings of the PSH or PSMC give the required storage and principal spillway capacity; the routings are discussed in chapter 17.

DEVELOPMENT OF TABLE 21.10. The principles of hydrograph development are discussed in chapter 16 but because the standard series of PSH and PSMC is not described there, the method of preparation will be briefly given here.

The PSH and PSMC in table 21.10 are developed from a continuous 10-day period of on-site direct runoff, all of a given frequency. Choice of the 10-day period is based on SCS experience with the use of both stream-flow records and an earlier system of standardized hydrographs. If the runoff in the 10-day period is arranged in order of decreasing

rate of flow and then accumulated to form a mass curve, it has the appearance of curve A in figure 21.1. Such a curve is a straight line on log paper and it has the equation:

$$Q_D = Q_{10} \, (D/10)^a \qquad (21.2)$$

where Q_D = total runoff at time D in days
Q_{10} = total runoff at the end of 10 days
D = time in days
a = log (Q_{10}/Q_1), in which Q_1 is the total runoff at the end of 1 day

Thus, knowing only the 1- and 10-day runoff amounts, a continuous mass curve can be developed for the entire 10-day period.

Examination of such mass curves of runoff from streamflow stations in many locations of the United States showed that the exponent <u>a</u> varied from 0.1 to 0.5. Extremes of 0.0458 and 0.699 were chosen for the standard curves; these extremes correspond to Q_1/Q_{10} ratios of 0.9 and 0.2 respectively. The ratio Q_1/Q_{10} is used hereafter in this chapter as a parameter in preference to <u>a</u> or Q_{10}/Q_1 because Q_{10} is more satisfactory as a divisor in preparing PSH and PSMC with dimensionless rates and amounts of flow. Q_1/Q_{10} ratios of 0.2, 0.3, 0.4, 0.5, 0.6, 0.7, 0.8, and 0.9 were selected to give representative degrees of curvature for the runoff curves.

The 10-day on-site runoff for each Q_1/Q_{10} ratio was rearranged as shown in table 21.5 to provide a moderately critical distribution of the 10-day runoff. This gave a distribution midway between extremes that are theoretically possible. On figure 21.1, curves A and B show the extremes and curve C shows the rearranged distribution for a Q_1/Q_{10} ratio of 0.4.

The effects of watershed lag were included by taking increments of runoff for each of the eight typical mass curves, making incremental hydrographs, and summing these to give total hydrographs for watersheds with times of concentration of 1.5, 3, 6, 12, 18, 24, 30, 36, 42, 48, 54, 60, 66, and 72 hours. This gave 112 hydrographs, each of which was reduced to unit rates of runoff and afterwards accumulated and reduced to unit mass curves. Curve D in figure 21.1 is the mass curve developed from curve C for a watershed with a time of concentration of 24 hours. Runoff for curve D went on for more than a day past the termination point E but because the rate was so small, the mass curve was terminated as shown. Other PSH and PSMC in table 21.10 are similarly terminated. The time interval is varied to reduce the size of the table and at the same time give enough points for reproducing the PSH and PSMC accurately. Straight-line connection of points is accurate enough for graphical work and linear interpolation for tabular work.

<u>USE OF TABLE 21.10</u>. The parameters for selecting a set of tabulations from table 21.10 are the Q_1/Q_{10} ratio and the time of concentration T_c in hours. The ratio and T_c of a watershed will seldom be values for

Table 21.5.--Arrangement of increments before construction of PSH and PSMC

Time	Increment
days	
0.0 to 0.5	19th largest 1/2 day
0.5 to 1.0	17th " " "
1.0 to 1.5	15th " " "
1.5 to 2.0	13th " " "
2.0 to 2.5	11th " " "
2.5 to 3.0	9th " " "
3.0 to 3.5	7th " " "
3.5 to 4.0	5th " " "
4.0 to 4.5	3rd " " "
4.5 to 4.6	9th largest 1/10 day
4.6 to 4.7	7th " " "
4.7 to 4.8	5th " " "
4.8 to 4.9	3rd " " "
4.9 to 5.0	Largest 1/10 day
5.0 to 5.1	2nd largest 1/10 day
5.1 to 5.2	4th " " "
5.2 to 5.3	6th " " "
5.3 to 5.4	8th " " "
5.4 to 5.5	10th " " "
5.5 to 6.0	4th largest 1/2 day
6.0 to 6.5	6th " " "
6.5 to 7.0	8th " " "
7.0 to 7.5	10th " " "
7.5 to 8.0	12th " " "
8.0 to 8.5	14th " " "
8.5 to 9.0	16th " " "
9.0 to 9.5	18th " " "
9.5 to 10.0	20th " " "

which the table is prepared, therefore choose that set having a Q_1/Q_{10} ratio and T_c nearest those of the watershed. It is easier to make the choice on table 21.9, which gives available PSH and PSMC and their serial numbers, and then to look up the serial number in table 21.10 for the tabulations.

Examples

The procedure by which a PSH or PSMC is developed will be illustrated by four examples. In example 21.1, channel losses are taken from direct runoff before development of a PSH and PSMC; in example 21.2, QRF is added to a PSH and PSMC; in example 21.3, runoff volume and rate maps (exhibit 21.1 through 21.5) are used to obtain runoff; and in example 21.4, upstream releases are added to a PSH.

Example 21.1.--Develop the 50-year frequency PSH and PSMC for a watershed located at latitude __________, longitude__________. The watershed has a drainage area of 15.0 square miles, time of concentration of 7.1 hours, average annual precipitation of 22.8 inches, average annual temperature of 61.5°F, and a runoff curve number (CN) of 80. There are no upstream structures.

1. Compile the 1- and 10-day point rainfall amounts from U.S. Weather Bureau maps. For this location TP-40 and TP-49 are used. The 50-year frequency 1- and 10-day amounts are 6.8 and 11.0 inches respectively.

2. Determine the areal rainfall. Get the adjustment factors from table 21.1. For the drainage area of 15.0 square miles they are 0.978 and 0.991 for the 1- and 10-day rains respectively. The areal rainfall is 0.978(6.8) = 6.65 inches for the 1-day rain and 0.991(11.0) = 10.9 inches for the 10-day rain.

3. Determine the CN for the 10-day rain. First check whether the 100-year frequency 10-day point rainfall amount is 6 or more inches. The appropriate map in TP-49 shows it is, therefore enter table 21.2 with the 1-day CN of 80 and find the 10-day CN is 65.

4. Estimate the direct runoff for 1 and 10 days. Enter figure 10.1 with the rainfall amounts from step 2 and the appropriate CN from step 3 and find $Q_1 = 4.37$ and $Q_{10} = 6.34$ inches.

5. Compute the climatic index. Using the given data and equation 21.1, the index Ci is $100(22.8)/61.5^2 = 0.603$. Because the Ci is less than 1 the channel loss may be used to reduce direct runoff.

6. Estimate the net runoff. The net runoff is the direct runoff minus the channel loss but when table 21.3 is used the net runoff is obtained by a multiplication not a subtraction. Enter

table 21.3 with the drainage area 15.0 square miles and the Ci of 0.603 and by interpolation find a reduction factor of 0.75. Multiply Q_1 and Q_{10} of step 4 by the factor to get net runoffs of 3.28 and 4.76 inches respectively. The net runoffs will be Q_1 and Q_{10} in the rest of this example.

7. Compute the Q_1/Q_{10} ratio. From step 6, Q_1/Q_{10} = 3.28/4.76 = 0.689.

8. Find the PSH and PSMC tabulations in table 21.10. Enter table 21.9 with the ratio 0.689 and T_c of 7.1 hours and find that the PSH with values nearest those is No. 22. Locate the appropriate tabulations in table 21.10 by looking up PSH No. 22. Columns 1, 2, and 4 of table 21.6 show the time, rate, and mass tabulations taken from table 21.10.

9. Compute PSH discharges in cfs. First find the product of drainage area and Q_{10}. This is 15.0(4.76) = 71.40 mile^2-inches. Multiply the entries in column 2, table 21.6 by 71.40, to get the discharges in cfs in column 3.

10. Compute PSMC amounts in inches. Multiply the entries in column 4, table 21.6, by Q_{10} (4.76) to get accumulated runoff in inches as shown in column 5. If amounts in acre-feet or another unit are desired, convert Q_{10} to the desired unit before making the series of multiplications.

The example is completed with step 10. The next step is that of routing the PSH or PSMC through the structure; see chapter 17 for routing methods.

In the second example the steps concerning channel loss are omitted and steps concerning QRF are included.

Example 21.2--Develop the 25-year frequency PSH and PSMC for a watershed at latitude ________, longitude ________. The watershed has a drainage area of 8.0 square miles, time of concentration of 2.0 hours, average annual precipitation of 30.5 inches, average annual temperature of 53.1°F, and a runoff curve number of 75. QRF during flood periods is estimated to be 5 cfs. There are no upstream structures in the watershed.

1. Compile the 1- and 10-day point rainfall amounts from U.S. Weather Bureau maps. For this location TP-40 and TP-49 are used. The 25-year frequency 1- and 10-day amounts are 5.6 and 12.5 inches respectively.

2. Determine the areal rainfall. Because the drainage area is not over 10 square miles the areal rainfall is the same as the point rainfall. The amounts in step 1 will be used.

Table 21.6.--PSH and PSMC for example 21.1

Time	$\frac{\text{cfs}}{A\ Q_{10}}$	PSH	$\frac{\text{Acc. Q}}{Q_{10}}$	PSMC
days	csm/inch	cfs		inches
0	0	0	0	0
.2	.231	16	.0007	.00
.5	.418	30	.0045	.02
1.0	.535	38	.0135	.06
2.0	.610	44	.0340	.16
3.0	.837	60	.0609	.29
3.6	1.123	80	.0827	.39
4.0	1.398	100	.1019	.48
4.3	1.932	138	.1196	.57
4.6	2.865	204	.1464	.70
4.8	3.973	284	.1709	.81
4.9	5.461	390	.1883	.90
5.0	27.118	1936	.2482	1.18
5.1	55.278	3947	.3998	1.90
5.2	41.011	2928	.5770	2.75
5.3	23.735	1695	.6961	3.31
5.4	13.975	998	.7655	3.64
5.5	8.668	619	.8072	3.84
5.6	5.638	402	.8335	3.97
5.8	2.818	201	.8634	4.11
6.0	1.859	133	.8798	4.19
6.5	1.360	97	.9078	4.32
7.0	1.002	72	.9290	4.42
7.5	.804	57	.9453	4.50
8.0	.687	59	.9588	4.56
9.0	.533	38	.9812	4.67
9.9	.416	30	.9966	4.74
10.1	.194	14	.9990	4.76
10.3	.044	3	.9998	4.76
10.8	0	0	1.0000	4.76

3. Determine the CN for the 10-day rain. The 10-day amount in step 1 is over 6 inches therefore the 100-year 10-day amount is too, and table 21.2 may be used. Enter the table with the CN of 75 for 1 day and find the CN is 58 at 10 days.

4. Estimate the direct runoff for 1 and 10 days. Enter figure 10.1 with the rainfall amounts from step 2 and the appropriate CN from step 3 and find $Q_1 = 2.94$ and $Q_{10} = 6.68$ inches. Because there are no channel losses, the direct runoff is the net runoff.

5. Compute the Q_1/Q_{10} ratio. From step 4, $Q_1/Q_{10} = 2.94/6.68 = 0.440$.

6. Find the PSH and PSMC tabulations in table 21.10. Enter table 21.9 with the ratio of 0.440 and T_c of 2.0 hours and find that the PSH and PSMC with values nearest those is No. 3. Locate the appropriate tabulations in table 21.10 by looking up PSH No. 3.

7. Compute PSH discharges in cfs. First find the product of drainage area and Q_{10}. This is 8.0(6.68) = 53.44 mile^2-inches. Multiply the entries in table 21.10 for PSH No. 3 by 53.44 to get discharges in cfs. These are shown in column 2, table 21.7, under the heading of "Preliminary PSH" because the final PSH must contain QRF.

8. Compute PSMC amounts in inches. Multiply the entries in table 21.10 for PSMC No. 3 by Q_{10} (6.68 inches) to get accumulated runoff in inches. The results are shown in column 5, table 21.7, under the heading "Preliminary PSMC" because the final PSMC must contain accumulated QRF. If the PSMC is to be in acre-feet or another unit, convert Q_{10} to the desired unit before making the series of multiplications.

9. Determine the minimum permissible quick return flow. First compute the climatic index: using the average annual precipitation and temperature and equation 21.1, the index Ci is $100(30.5)/53.1^2 = 1.08$. Enter table 21.4 with the Ci of 1.08 and find that the minimum QRF is 0.045 inches per day or 1.20 csm, which converts to 8.0(1.20) = 9.6 cfs. The locally estimated QRF is 5 cfs. Therefore the minimum permissible QRF is 9.6 cfs because it is larger than the locally estimated flow. Round 9.6 to 10 cfs and tabulate in column 3, table 21.7.

10. Add QRF to the preliminary PSH. The QRF shown in column 3, table 21.7, is added to the preliminary PSH, column 2, to give the PSH discharges in column 4.

11. Add QRF to the preliminary PSMC. The accumulated QRF in inches, column 6, table 21.7, is added to the preliminary PSMC column 5, to give the PSMC amounts in column 7.

Table 21.7.--PSH and PSMC for example 21.2

Time	Prelim-inary PSH	QRF*	PSH	Prelim-inary PSMC	Acc. QRF**	PSMC
days	cfs	cfs	cfs	inches	inches	inches
0	0	10	10	0	0	0
.1	48	10	58	.01	.00	.01
.5	60	10	70	.11	.02	.13
1.0	69	10	79	.26	.04	.30
2.0	78	10	88	.60	.09	.69
3.0	100	10	110	1.00	.14	1.14
3.5	118	10	128	1.26	.16	1.42
4.0	146	10	156	1.58	.18	1.76
4.2	181	10	191	1.72	.19	1.91
4.4	230	10	240	1.91	.20	2.11
4.6	259	10	269	2.13	.21	2.34
4.7	298	10	308	2.25	.21	2.46
4.8	370	10	380	2.40	.22	2.62
4.9	512	10	522	2.60	.22	2.82
5.0	1992	10	2002	3.16	.22	3.38
5.1	1039	10	1049	3.84	.23	4.07
5.2	567	10	577	4.20	.23	4.43
5.3	383	10	393	4.42	.24	4.66
5.4	302	10	312	4.57	.24	4.81
5.5	257	10	267	4.69	.25	4.94
5.6	207	10	217	4.80	.25	5.05
5.8	174	10	184	4.97	.26	5.23
6.0	154	10	164	5.11	.27	5.38
6.5	128	10	138	5.41	.29	5.70
7.0	108	10	118	5.66	.32	5.98
8.0	84	10	94	6.07	.36	6.43
9.0	72	10	82	6.41	.40	6.81
10.0	57	10	67	6.66	.45	7.11
10.1	2	10	12	6.68	.45	7.13
10.3	0	10	10	6.68	.46	7.14
11.0	0	10	10	6.68	.50	7.18
12.0	0	10	10	6.68	.54	7.22
etc.	etc.	etc.	etc.	etc.	etc.	etc.

* 9.6 cfs rounded to 10 cfs.

** At a rate of 0.045 inches per day.

In the third example the use of the runoff volume maps is illustrated.

Example 21.3--Develop the 100-year frequency PSH for a watershed located at 43° latitude and 77° longitude. The watershed has a drainage area of 12 square miles, time of concentration of 3.5 hours.

1. Estimate 100-year 10-day runoff volumes from exhibit 21.1. The interpolated value is 8.8.

2. Select the Q_1/Q_{10} ratio from exhibit 21.2. For this area the value is 0.4.

3. Calculate 1-day volume of runoff. $Q_1/Q_{10} = 0.4$, $Q_1 = (0.4)(8.8) = 3.52$ inches.

4. Find the PSH tabulations in Table 21.10. Enter table 21.9 with the Q_1/Q_{10} ratio of 0.4 and Tc of 3.5 hours and find that the PSH with values nearest is No. 11. Locate appropriate tabulations in table 21.10 by looking up PSH No. 11.

5. Compute PSH discharges in cfs. Find the product of drainage area and Q_{10}. This is (12) (8.8) = 105.6 mile^2-inches. Entries for PSH No. 11 are multiplied by this value to obtain discharge in cfs. These are shown in column 2, table 21.8.

6. Determine the quick-return flow rate. From exhibit 21.3 the interpolated value is 5.3 csm.

7. Extension of quick-return flow rates beyond the PSH. The quick-return flow rate is (12) (5.3) = 63.6 cfs, round to 64 cfs. This constant rate of discharge is an extension to the PSH as shown in figure 21.1a, and column 4, table 21.8. No value less than 64 cfs should be used in the recession side of the PSH.

The procedure for adding releases from upstream structures is shown in the following descriptive example. If a lower structure has channel losses in its contributing area the deduction for channel loss is made in the preliminary PSH for that area. Deductions may also be required for PSH of the upper structures but once these PSH are routed through the structures no further deductions are made in the release rates.

Example 21.4--Adding releases from upstream structures when developing the PSH for a lower structure in a series is done as follows:

1. Develop the preliminary PSH for the lower structure. Use the method of example 21.1 or 21.2 or 21.3 whichever is applicable.

Table 21.8.--PSH for Example 21.3.

Time	Prelim-inary PSH	QRF	PSH
days	cfs	cfs	cfs
0	0		0
.1	61		61
.5	116		116
1.0	134		134
2.0	151		151
3.0	195		195
3.5	230		230
4.0	285		285
4.3	371		371
4.6	495		495
4.8	667		667
4.9	894		894
5.0	2885		2885
5.1	2455		2455
5.2	1478		1478
5.3	954		954
5.4	696		696
5.5	552		552
5.6	446		446
5.7	383		383
5.8	352		352
6.0	307		307
6.5	251		251
7.0	211		211
7.5	181		181
8.0	163		163
9.0	140		140
10.0	111		111
10.1	16	64	64
10.7	0	64	64
11.0	0	64	64
12.0	0	64	64
etc.	etc.	etc.	etc.

2. Flood-route the upstream structure releases or outflows to the lower structure. Chapter 17 discusses flood-routing procedures.

3. Add the routed flows to the preliminary PSH to get the PSH for the lower structure.

Note that if an upstream structure is itself a lower structure in a series then the procedure of example 21.4 must be followed for it first.

Table 21.9.--Serial numbers of PSH and PSMC

T_c	Q_1/Q_{10}							
	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9
hours	Serial numbers							
1.5*	1	2	3	4	5	6	7	8
3	9	10	11	12	13	14	15	16
6	17	18	19	20	21	22	23	24
12	25	26	27	28	29	30	31	32
18	33	34	35	36	37	38	39	40
24	41	42	43	44	45	46	47	48
30	49	50	51	52	53	54	55	56
36	57	58	59	60	61	62	63	64
42	65	66	67	68	69	70	71	72
48	73	74	75	76	77	78	79	80
54	81	82	83	84	85	86	87	88
60	89	90	91	92	93	94	95	96
66	97	98	99	100	101	102	103	104
72**	105	106	107	108	109	110	111	112

* Use this row for all T_c less than 1.5 hours.

** Use this row for all T_c over 72 hours.

Table 21.10.--Time, rate and mass tabulations for Principal Spillway Hydrographs (PSH) and Mass Curves (PSMC)

T_c = 1.5 hours

Serial No. :	1		2		3		4	
Q_1/Q_{10} :	0.2		0.3		0.4		0.5	
Time	PSH	PSMC	PSH	PSMC	PSH	PSMC	PSH	PSMC
days	cfs/AQ_{10}	Q/Q_{10}	cfs/AQ_{10}	Q/Q_{10}	cfs/AQ_{10}	Q/Q_{10}	cfs/AQ_{10}	Q/Q_{10}
0	0	0	0	0	0	0	0	0
.1	1.584	.0028	1.188	.0021	.890	.0016	.704	.0013
.5	2.014	.0308	1.510	.0230	1.119	.0170	.895	.0136
1.0	2.126	.0687	1.594	.0515	1.286	.0397	.951	.0305
2.0	2.237	.1480	1.846	.1156	1.454	.0894	1.203	.0705
3.0	2.517	.2358	2.209	.1904	1.873	.1505	1.510	.1208
3.5	2.741	.2845	2.489	.2342	2.208	.1890	1.846	.1530
4.0	3.210	.3385	2.992	.2866	2.741	.2365	2.405	.1946
4.2	3.470	.3624	3.618	.3094	3.394	.2583	3.222	.2144
4.4	3.760	.3885	4.237	.3374	4.313	.2854	3.928	.2396
4.6	4.060	.4172	4.732	.3701	4.851	.3186	4.655	.2706
4.7	4.342	.4323	5.257	.3881	5.570	.3373	5.485	.2888
4.8	4.868	.4489	6.209	.4087	6.916	.3597	6.966	.3111
4.9	5.708	.4679	8.068	.4343	9.587	.3893	10.303	.3421
5.0	10.027	.4962	21.540	.4876	37.270	.4734	57.224	.4632
5.1	7.689	.5281	13.395	.5504	19.442	.5752	25.499	.6115
5.2	5.825	.5524	8.470	.5897	10.603	.6291	12.108	.6790
5.3	4.916	.5718	6.320	.6162	7.162	.6610	7.460	.7141
5.4	4.444	.5886	5.270	.6371	5.642	.6840	5.520	.7373
5.5	4.065	.6040	4.652	.6549	4.812	.7027	4.584	.7555
5.6	3.546	.6176	3.976	.6704	3.875	.7183	3.605	.7701
5.8	3.300	.6430	3.230	.6971	3.261	.7435	2.847	.7927
6.0	3.193	.6659	3.124	.7196	2.882	.7653	2.553	.8121
6.5	2.797	.7183	2.713	.7696	2.405	.8100	2.070	.8505
7.0	2.629	.7661	2.321	.8126	2.020	.8476	1.678	.8816
8.0	2.293	.8526	1.846	.8848	1.566	.9082	1.230	.9305
9.0	2.126	.9306	1.594	.9458	1.342	.9590	.951	.9683
10.0	1.902	.9948	1.510	.9959	1.063	.9971	.839	.9977
10.1	.070	.9998	.056	.9999	.039	.9999	.031	.9999
10.3	0	1.0000	0	1.0000	0	1.0000	0	1.0000

Table 21.10.--(Continued)

T_c = 1.5 hours

Serial No. :	5		6		7		8	
Q_1/Q_{10} :	0.6		0.7		0.8		0.9	
Time	PSH	PSMC	PSH	PSMC	PSH	PSMC	PSH	PSMC
days	cfs/AQ_{10}	Q/Q_{10}	cfs/AQ_{10}	Q/Q_{10}	cfs/AQ_{10}	Q/Q_{10}	cfs/AQ_{10}	Q/Q_{10}
0	0	0	0	0	0	0	0	0
.1	.528	.0009	.352	.0006	.198	.0004	.088	.0002
.5	.671	.0102	.470	.0068	.280	.0040	.140	.0019
1.0	.754	.0232	.559	.0164	.330	.0095	.168	.0047
2.0	.922	.0534	.642	.0373	.442	.0240	.218	.0113
3.0	1.225	.0929	.867	.0654	.587	.0428	.302	.0203
3.5	1.482	.1186	1.113	.0844	.671	.0546	.390	.0268
4.0	2.014	.1533	1.454	.1095	1.062	.0723	.531	.0359
4.2	2.808	.1702	2.034	.1222	1.650	.0826	.838	.0412
4.4	3.374	.1918	2.855	.1400	1.678	.0946	.974	.0479
4.6	4.154	.2191	3.405	.1621	2.442	.1096	1.270	.0555
4.7	4.960	.2354	4.162	.1757	3.055	.1194	1.660	.0607
4.8	6.567	.2561	5.627	.1932	4.179	.1324	2.317	.0678
4.9	10.131	.2860	9.071	.2195	6.888	.1522	3.956	.0790
5.0	81.384	.4500	109.748	.4323	142.265	.4191	179.016	.4063
5.1	31.367	.6520	36.714	.6945	41.728	.7483	45.898	.8086
5.2	12.872	.7312	13.042	.7836	12.441	.8452	11.085	.9105
5.3	7.150	.7671	6.332	.8183	5.140	.8767	3.430	.9364
5.4	5.069	.7890	4.242	.8372	3.117	.8915	1.704	.9456
5.5	4.112	.8054	3.366	.8508	2.426	.9014	1.298	.9510
5.6	2.998	.8182	2.554	.8614	1.696	.9088	.909	.9550
5.8	2.554	.8379	1.976	.8770	1.406	.9195	.805	.9605
6.0	2.028	.8543	1.622	.8897	1.088	.9286	.569	.9652
6.5	1.678	.8853	1.371	.9152	.929	.9459	.426	.9734
7.0	1.342	.9103	1.007	.9344	.671	.9586	.314	.9796
8.0	.924	.9481	.699	.9626	.420	.9765	.224	.9887
9.0	.727	.9769	.532	.9840	.308	.9897	.168	.9953
10.0	.587	.9984	.420	.9989	.258	.9993	.118	.9997
10.1	.022	1.0000	.016	1.0000	.009	1.0000	.004	1.0000
10.3	0	1.0000	0	1.0000	0	1.0000	0	1.0000

Table 21.10.--(Continued)

T_c = 3 hours

Serial No.:	9		10		11		12	
Q_1/Q_{10} :	0.2		0.3		0.4		0.5	
Time	PSH	PSMC	PSH	PSMC	PSH	PSMC	PSH	PSMC
days	cfs/AQ_{10}	Q/Q_{10}	cfs/AQ_{10}	Q/Q_{10}	cfs/AQ_{10}	Q/Q_{10}	cfs/AQ_{10}	Q/Q_{10}
0	0	0	0	0	0	0	0	0
.1	1.034	.0019	.775	.0014	.574	.0010	.460	.0008
.5	1.984	.0277	1.488	.0207	1.102	.0153	.882	.0122
1.0	2.097	.0654	1.572	.0490	1.269	.0377	.938	.0290
2.0	2.207	.1445	1.821	.1128	1.434	.0872	1.186	.0686
3.0	2.483	.2319	2.178	.1870	1.844	.1476	1.490	.1185
3.5	2.703	.2803	2.455	.2304	2.175	.1856	1.820	.1501
4.0	3.226	.3336	2.951	.2819	2.702	.2322	2.372	.1909
4.3	3.515	.3697	3.687	.3172	3.516	.2657	3.283	.2214
4.6	3.982	.4110	4.599	.3630	4.687	.3114	4.455	.2638
4.8	4.607	.4419	5.770	.4001	6.321	.3505	6.315	.3020
4.9	5.310	.4600	7.265	.4238	8.462	.3774	8.934	.3296
5.0	8.383	.4850	16.609	.4674	27.323	.4424	40.542	.4196
5.1	8.061	.5150	15.002	.5250	23.244	.5344	32.577	.5526
5.2	6.429	.5414	10.246	.5710	13.995	.6022	17.510	.6436
5.3	5.305	.5628	7.384	.6031	9.038	.6441	10.235	.6940
5.4	4.654	.5810	5.842	.6272	6.587	.6725	6.862	.7251
5.5	4.194	.5972	4.926	.6468	5.225	.6940	5.100	.7468
5.6	3.708	.6116	4.214	.6635	4.227	.7112	3.989	.7634
5.7	3.583	.6249	3.874	.6782	3.631	.7255	3.293	.7766
5.8	3.367	.6376	3.406	.6915	3.331	.7382	2.940	.7880
6.0	3.143	.6610	3.095	.7148	2.905	.7607	2.581	.8079
6.5	2.762	.7140	2.677	.7654	2.374	.8063	2.042	.8473
7.0	2.593	.7620	2.291	.8090	2.000	.8444	1.656	.8790
7.5	2.428	.8071	2.069	.8477	1.712	.8770	1.407	.9057
8.0	2.262	.8490	1.821	.8819	1.545	.9058	1.214	.9286
9.0	2.097	.9273	1.573	.9433	1.324	.9569	.938	.9669
10.0	1.877	.9919	1.490	.9936	1.050	.9955	.829	.9964
10.1	.280	.9991	.222	.9993	.156	.9995	.123	.9996
10.7	0	1.0000	0	1.0000	0	1.0000	0	1.0000

Table 21.10.--(Continued)

T_c = 3 hours

Serial No. :	13		14		15		16	
Q_1/Q_{10} :	0.6		0.7		0.8		0.9	
Time	PSH	PSMC	PSH	PSMC	PSH	PSMC	PSH	PSMC
days	cfs/AQ_{10}	Q/Q_{10}	cfs/AQ_{10}	Q/Q_{10}	cfs/AQ_{10}	Q/Q_{10}	cfs/AQ_{10}	Q/Q_{10}
0	0	0	0	0	0	0	0	0
.1	.345	.0006	.230	.0004	.129	.0002	.057	.0001
.5	.661	.0092	.455	.0061	.274	.0036	.137	.0017
1.0	.741	.0221	.550	.0156	.318	.0090	.165	.0045
2.0	.906	.0520	.630	.0363	.428	.0234	.208	.0110
3.0	1.200	.0910	.855	.0641	.579	.0420	.290	.0198
3.5	1.462	.1164	1.090	.0827	.662	.0536	.382	.0262
4.0	1.986	.1502	1.434	.1073	1.044	.0707	.524	.0351
4.3	2.802	.1762	2.305	.1270	1.626	.0860	.892	.0431
4.6	3.961	.2131	3.220	.1573	2.277	.1062	1.160	.0538
4.8	5.881	.2477	5.004	.1861	3.699	.1271	2.035	.0650
4.9	8.682	.2741	7.686	.2091	5.803	.1444	3.303	.0746
5.0	56.240	.3920	74.415	.3581	94.971	.3272	118.066	.2947
5.1	42.862	.5720	53.883	.5910	65.740	.6187	78.137	.6504
5.2	20.664	.6874	23.462	.7314	25.834	.7848	27.664	.8423
5.3	10.890	.7447	11.095	.7941	10.896	.8514	10.182	.9109
5.4	6.744	.7767	6.234	.8256	5.412	.8810	4.240	.9370
5.5	4.686	.7975	3.953	.8441	2.980	.8962	1.764	.9479
5.6	3.438	.8122	2.890	.8565	1.996	.9053	1.073	.9531
5.7	2.871	.8237	2.282	.8659	1.580	.9118	.793	.9564
5.8	2.618	.8337	2.033	.8737	1.436	.9172	.781	.9593
6.0	2.113	.8509	1.659	.8870	1.149	.9267	.587	.9642
6.5	1.656	.8827	1.356	.9130	.924	.9445	.427	.9728
7.0	1.325	.9082	.995	.9328	.662	.9576	.317	.9791
7.5	1.080	.9291	.802	.9484	.525	.9678	.250	.9841
8.0	.915	.9467	.690	.9615	.414	.9759	.221	.9883
9.0	.719	.9758	.528	.9832	.304	.9892	.166	.9951
10.0	.582	.9975	.415	.9982	.262	.9989	.123	.9995
10.1	.086	.9997	.062	.9998	.038	.9999	.018	.9999
10.7	0	1.0000	0	1.0000	0	1.0000	0	1.0000

Table 21.10.--(Continued)

T_c = 6 hours

Serial No. :	17		18		19		20	
Q_1/Q_{10} :	0.2		0.3		0.4		0.5	
Time	PSH	PSMC	PSH	PSMC	PSH	PSMC	PSH	PSMC
days	cfs/AQ_{10}	Q/Q_{10}	cfs/AQ_{10}	Q/Q_{10}	cfs/AQ_{10}	Q/Q_{10}	cfs/AQ_{10}	Q/Q_{10}
0	0	0	0	0	0	0	0	0
.2	1.038	.0031	.779	.0023	.577	.0017	.461	.0014
.5	1.862	.0205	1.397	.0154	1.035	.0114	.828	.0091
1.0	2.063	.0575	1.547	.0431	1.244	.0329	.923	.0255
2.0	2.174	.1361	1.792	.1059	1.410	.0818	1.164	.0641
3.0	2.444	.2225	2.136	.1787	1.800	.1407	1.462	.1128
3.6	2.714	.2800	2.489	.2302	2.215	.1854	1.876	.1500
4.0	3.006	.3220	2.886	.2709	2.636	.2222	2.314	.1820
4.3	3.284	.3571	3.349	.3044	3.178	.2536	2.944	.2102
4.6	3.801	.3964	4.282	.3466	4.310	.2950	4.029	.2485
4.8	4.196	.4258	5.046	.3807	5.340	.3300	5.225	.2820
4.9	4.653	.4421	5.951	.4010	6.616	.3521	6.721	.3040
5.0	5.991	.4618	9.630	.4298	13.534	.3892	17.748	.3491
5.1	7.547	.4868	14.087	.4736	22.175	.4551	31.771	.4404
5.2	7.180	.5141	12.665	.5230	18.923	.5309	25.805	.5464
5.3	6.166	.5388	9.785	.5645	13.444	.5906	17.306	.6254
5.4	5.330	.5601	7.628	.5967	9.677	.6332	11.430	.6778
5.5	4.723	.5786	6.186	.6222	7.310	.6645	8.067	.7138
5.6	4.212	.5952	5.169	.6432	5.727	.6886	5.954	.7396
5.8	3.587	.6237	3.923	.6764	3.881	.7233	3.641	.7741
6.0	3.188	.6486	3.214	.7023	3.109	.7487	2.784	.7972
6.5	2.757	.7034	2.662	.7552	2.372	.7971	2.040	.8394
7.0	2.566	.7522	2.282	.8002	2.000	.8367	1.652	.8727
7.5	2.403	.7978	2.052	.8398	1.706	.8704	1.400	.9003
8.0	2.240	.8404	1.808	.8750	1.532	.8999	1.207	.9239
9.0	2.071	.9193	1.559	.9373	1.312	.9519	.933	.9633
9.9	1.862	.9847	1.475	.9879	1.052	.9914	.828	.9932
10.1	.872	.9955	.692	.9965	.490	.9975	.386	.9980
10.3	.198	.9991	.158	.9992	.111	.9995	.040	.9998
10.8	0	1.0000	0	1.0000	0	1.0000	0	1.0000

Table 21.10.--(Continued)

T_c = 6 hours

Serial No. :	21		22		23		24	
Q_1/Q_{10} :	0.6		0.7		0.8		0.9	
Time	PSH	PSMC	PSH	PSMC	PSH	PSMC	PSH	PSMC
days	cfs/AQ_{10}	Q/Q_{10}	cfs/AQ_{10}	Q/Q_{10}	cfs/AQ_{10}	Q/Q_{10}	cfs/AQ_{10}	Q/Q_{10}
0	0	0	0	0	0	0	0	0
.2	.346	.0010	.231	.0007	.130	.0004	.058	.0002
.5	.621	.0068	.418	.0045	.254	.0026	.124	.0012
1.0	.719	.0193	.535	.0135	.302	.0079	.160	.0039
2.0	.881	.0486	.610	.0340	.412	.0218	.194	.0102
3.0	1.167	.0865	.837	.0609	.566	.0398	.274	.0188
3.6	1.518	.1163	1.123	.0827	.708	.0536	.395	.0262
4.0	1.934	.1428	1.398	.1019	1.004	.0668	.510	.0331
4.3	2.527	.1666	1.932	.1196	1.489	.0804	.784	.0401
4.6	3.539	.1997	2.865	.1464	1.961	.0987	.999	.0500
4.8	4.747	.2295	3.973	.1709	2.887	.1161	1.555	.0591
4.9	6.335	.2499	5.461	.1883	4.056	.1289	2.255	.0661
5.0	22.276	.3026	27.118	.2482	32.166	.1955	37.622	.1394
5.1	42.826	.4225	55.278	.3998	69.093	.3817	84.295	.3634
5.2	33.204	.5625	41.011	.5770	49.241	.5993	57.738	.6245
5.3	20.462	.6613	23.735	.6961	26.833	.7392	29.654	.7851
5.4	12.851	.7226	13.975	.7655	14.846	.8159	15.379	.8679
5.5	8.521	.7619	8.668	.8072	8.572	.8589	8.194	.9112
5.6	5.896	.7885	5.638	.8335	5.120	.8841	4.424	.9344
5.8	3.326	.8212	2.818	.8634	2.199	.9096	1.490	.9546
6.0	2.389	.8417	1.859	.8798	1.326	.9216	.680	.9616
6.5	1.655	.8764	1.360	.9078	.931	.9409	.438	.9711
7.0	1.322	.9031	1.002	.9290	.666	.9551	.327	.9779
7.5	1.085	.9249	.804	.9453	.525	.9658	.253	.9832
8.0	.918	.9431	.687	.9588	.415	.9743	.221	.9875
9.0	.718	.9730	.533	.9812	.305	.9880	.165	.9944
9.9	.586	.9952	.416	.9966	.271	.9978	.129	.9990
10.1	.272	.9986	.194	.9990	.122	.9988	.057	.9997
10.3	.062	.9997	.044	.9998	.028	.9999	.013	.9999
10.8	0	1.0000	0	1.0000	0	1.0000	0	1.0000

Table 21.10.--(Continued)

T_c = 12 hours

Serial No. :	25		26		27		28	
Q_1/Q_{10} :	0.2		0.3		0.4		0.5	
Time	PSH	PSMC	PSH	PSMC	PSH	PSMC	PSH	PSMC
days	cfs/AQ_{10}	Q/Q_{10}	cfs/AQ_{10}	Q/Q_{10}	cfs/AQ_{10}	Q/Q_{10}	cfs/AQ_{10}	Q/Q_{10}
0	0	0	0	0	0	0	0	0
.3	.678	.0026	.509	.0019	.377	.0014	.302	.0011
.6	1.577	.0158	1.183	.0118	.879	.0088	.701	.0070
1.0	1.967	.0426	1.475	.0319	1.165	.0242	.878	.0189
2.0	2.156	.1198	1.764	.0926	1.379	.0714	1.124	.0557
3.0	2.408	.2043	2.075	.1631	1.726	.1278	1.414	.1022
4.0	2.842	.3006	2.748	.2502	2.486	.2035	2.164	.1658
4.3	3.105	.3336	2.992	.2818	2.979	.2325	2.507	.1913
4.6	3.485	.3701	3.711	.3187	3.630	.2677	3.345	.2234
4.8	3.804	.3971	4.310	.3483	4.377	.2971	4.148	.2508
4.9	4.043	.4116	4.768	.3651	4.995	.3144	4.855	.2674
5.0	4.540	.4275	5.944	.3849	6.976	.3365	7.736	.2907
5.1	5.388	.4459	8.174	.4110	11.052	.3698	14.079	.3309
5.2	6.200	.4673	10.329	.4452	15.007	.4179	20.236	.3942
5.3	6.451	.4908	10.879	.4844	15.865	.4749	21.358	.4710
5.4	6.163	.5141	9.984	.5230	14.080	.5302	18.384	.5443
5.5	5.659	.5360	8.609	.5574	11.562	.5776	14.463	.6049
5.6	5.157	.5561	7.374	.5870	9.437	.6164	11.327	.6525
5.8	4.298	.5910	5.483	.6342	6.345	.6741	7.000	.7192
6.0	3.706	.6205	4.796	.6533	4.558	.7138	4.649	.7615
6.2	3.331	.6465	3.500	.6985	3.519	.7434	3.366	.7907
6.5	2.940	.6812	2.893	.7335	2.684	.7772	2.389	.8220
6.8	2.717	.7126	2.569	.7638	2.286	.8046	1.948	.8457
7.4	2.477	.7702	2.161	.8159	1.848	.8502	1.519	.8837
8.0	2.283	.8232	1.875	.8608	1.582	.8880	1.262	.9144
9.0	2.086	.9036	1.601	.9253	1.341	.9418	.977	.9559
10.0	1.826	.9772	1.439	.9820	1.053	.9870	.822	.9898
10.3	.844	.9926	.667	.9942	.480	.9958	.377	.9967
10.6	.239	.9981	.189	.9985	.136	.9989	.107	.9991
11.4	0	1.0000	0	1.0000	0	1.0000	0	1.0000

Table 21.10.--(Continued)

T_c = 12 hours

Serial No. :	29		30		31		32	
Q_1/Q_{10} :	0.6		0.7		0.8		0.9	
Time	PSH	PSMC	PSH	PSMC	PSH	PSMC	PSH	PSMC
days	cfs/AQ_{10}	Q/Q_{10}	cfs/AQ_{10}	Q/Q_{10}	cfs/AQ_{10}	Q/Q_{10}	cfs/AQ_{10}	Q/Q_{10}
0	0	0	0	0	0	0	0	0
.3	.226	.0008	.151	.0006	.086	.0003	.038	.0001
.6	.526	.0052	.356	.0035	.212	.0020	.102	.0010
1.0	.672	.0142	.490	.0098	.281	.0058	.145	.0028
2.0	.847	.0423	.585	.0296	.403	.0188	.186	.0089
3.0	1.120	.0781	.801	.0549	.539	.0358	.259	.0169
4.0	1.794	.1294	1.303	.0922	.902	.0601	.470	.0296
4.3	2.121	.1507	1.574	.1078	1.197	.0714	.622	.0354
4.6	2.882	.1780	2.315	.1290	1.594	.0868	.848	.0436
4.8	3.671	.2020	2.999	.1484	2.114	.1002	1.112	.0507
4.9	4.396	.2169	3.664	.1607	2.644	.1090	1.421	.0554
5.0	8.270	.2402	8.608	.1833	8.709	.1299	8.691	.0740
5.1	17.276	.2873	20.646	.2372	24.136	.1904	27.865	.1412
5.2	25.994	.3671	32.253	.3347	38.973	.3066	46.207	.2776
5.3	27.302	.4653	33.657	.4561	40.402	.4527	47.511	.4500
5.4	22.834	.5577	27.414	.5686	32.115	.5862	36.878	.6053
5.5	17.279	.6317	20.012	.6560	22.676	.6871	25.213	.7196
5.6	13.048	.6876	14.617	.7198	16.047	.7584	17.313	.7978
5.8	7.474	.7620	7.808	.8007	7.959	.8447	7.993	.8884
6.0	4.661	.8058	4.506	.8450	4.272	.8884	3.938	.9308
6.2	3.122	.8341	2.813	.8714	2.431	.9125	1.968	.9518
6.5	2.029	.8618	1.724	.8957	1.290	.9323	.795	.9664
6.8	1.582	.8814	1.271	.9119	.858	.9436	.413	.9723
7.4	1.203	.9119	.907	.9355	.598	.9594	.294	.9800
8.0	.972	.9358	.724	.9534	.450	.9709	.234	.9857
9.0	.752	.9674	.560	.9770	.330	.9855	.174	.9932
10.0	.591	.9928	.415	.9949	.269	.9967	.125	.9985
10.3	.268	.9977	.189	.9984	.121	.9990	.056	.9995
10.6	.076	.9994	.054	.9996	.034	.9997	.016	.9999
11.4	0	1.0000	0	1.0000	0	1.0000	0	1.0000

Table 21.10.--(Continued)

T_c = 18 hours

Serial No. :	33		34		35		36	
Q_1/Q_{10} :	0.2		0.3		0.4		0.5	
Time	PSH	PSMC	PSH	PSMC	PSH	PSMC	PSH	PSMC
days	cfs/AQ_{10}	Q/Q_{10}	cfs/AQ_{10}	Q/Q_{10}	cfs/AQ_{10}	Q/Q_{10}	cfs/AQ_{10}	Q/Q_{10}
0	0	0	0	0	0	0	0	0
.3	.277	.0010	.208	.0007	.154	.0005	.123	.0004
.6	1.095	.0086	.821	.0064	.609	.0047	.487	.0038
1.0	1.736	.0302	1.302	.0226	1.008	.0170	.774	.0134
2.0	2.124	.1039	1.716	.0798	1.334	.0614	1.070	.0478
3.0	2.359	.1867	2.004	.1482	1.641	.1156	1.350	.0922
4.0	2.736	.2802	2.576	.2311	2.298	.1866	1.973	.1514
4.5	3.134	.3343	3.092	.2828	2.905	.2337	2.615	.1927
4.9	3.693	.3845	4.116	.3354	4.156	.2848	3.928	.2397
5.0	3.970	.3987	4.720	.3518	5.096	.3019	5.209	.2566
5.1	4.410	.4142	5.777	.3712	6.896	.3241	7.862	.2807
5.2	4.978	.4316	7.206	.3952	9.422	.3542	11.690	.3168
5.3	5.502	.4510	8.529	.4243	11.765	.3933	15.235	.3665
5.4	5.792	.4719	9.213	.4571	12.920	.4389	16.904	.4258
5.5	5.789	.4934	9.122	.4910	12.668	.4862	16.399	.4872
5.6	5.571	.5144	8.512	.5237	11.530	.5309	14.598	.5444
5.7	5.242	.5345	7.676	.5536	10.043	.5707	12.343	.5941
5.8	4.892	.5532	6.849	.5805	8.640	.6052	10.299	.6359
5.9	4.566	.5708	6.122	.6045	7.463	.6350	8.651	.6709
6.0	4.266	.5871	5.472	.6259	6.451	.6607	7.259	.7003
6.2	3.773	.6168	4.430	.6624	4.898	.7023	5.185	.7458
6.4	3.413	.6434	3.726	.6924	3.888	.7346	3.907	.7791
6.7	3.022	.6790	3.078	.7299	2.972	.7721	2.779	.8155
7.0	2.777	.7112	2.671	.7617	2.456	.8020	2.176	.8427
7.4	2.570	.7507	2.306	.7983	2.016	.8348	1.681	.8708
8.0	2.352	.8054	1.978	.8458	1.672	.8753	1.352	.9041
9.0	2.117	.8876	1.662	.9127	1.388	.9313	1.040	.9480
10.0	1.907	.9627	1.491	.9707	1.134	.9784	.874	.9832
10.3	1.375	.9816	1.082	.9855	.797	.9894	.620	.9917
10.7	.464	.9944	.366	.9956	.268	.9968	.209	.9975
11.0	.190	.9979	.149	.9983	.109	.9988	.085	.9990
12.0	0	1.0000	0	1.0000	0	1.0000	0	1.0000

Table 21.10.--(Continued)

T_c = 18 hours

Serial No. :	37		38		39		40	
Q_1/Q_{10} :	0.6		0.7		0.8		0.9	
Time	PSH	PSMC	PSH	PSMC	PSH	PSMC	PSH	PSMC
days	cfs/AQ_{10}	Q/Q_{10}	cfs/AQ_{10}	Q/Q_{10}	cfs/AQ_{10}	Q/Q_{10}	cfs/AQ_{10}	Q/Q_{10}
0	0	0	0	0	0	0	0	0
.3	.092	.0003	.062	.0002	.035	.0001	.016	.0000
.6	.365	.0028	.245	.0019	.144	.0011	.068	.0005
1.0	.588	.0101	.418	.0069	.244	.0040	.122	.0019
2.0	.809	.0363	.561	.0254	.387	.0159	.177	.0076
3.0	1.059	.0703	.754	.0494	.506	.0320	.241	.0151
4.0	1.616	.1176	1.179	.0836	.789	.0544	.417	.0266
4.5	2.214	.1520	1.684	.1090	1.228	.0724	.647	.0360
4.9	3.472	.1925	2.830	.1410	2.004	.0951	1.065	.0480
5.0	5.102	.2084	4.811	.1551	4.331	.1068	3.740	.0568
5.1	8.709	.2338	9.462	.1814	10.106	.1334	10.725	.0835
5.2	14.028	.2757	16.442	.2292	18.910	.1868	21.507	.1428
5.3	18.934	.3365	22.854	.3016	26.967	.2713	31.324	.2400
5.4	21.138	.4104	25.608	.3909	30.294	.3767	35.205	.3625
5.5	20.281	.4868	24.302	.4828	28.455	.4849	32.720	.4875
5.6	17.690	.5568	20.808	.5659	23.951	.5814	27.092	.5976
5.7	14.565	.6162	16.726	.6351	18.819	.6602	20.835	.6858
5.8	11.834	.6649	13.268	.6904	14.589	.7217	15.811	.7533
5.9	9.716	.7046	10.671	.7345	11.506	.7698	12.251	.8049
6.0	7.960	.7372	8.536	.7699	9.005	.8075	9.384	.8447
6.2	5.384	.7860	5.469	.8210	5.475	.8602	5.391	.8984
6.4	3.847	.8197	3.751	.8545	3.565	.8930	3.308	.9299
6.7	2.526	.8542	2.311	.8872	1.978	.9227	1.586	.9558
7.0	1.873	.8781	1.609	.9084	1.260	.9401	.881	.9689
7.4	1.350	.9016	1.039	.9275	.694	.9540	.341	.9774
8.0	1.051	.9278	.788	.9474	.503	.9671	.254	.9838
9.0	.795	.9613	.592	.9725	.361	.9828	.184	.9918
10.0	.640	.9879	.446	.9915	.288	.9946	.131	.9975
10.3	.447	.9941	.314	.9958	.202	.9973	.094	.9988
10.7	.150	.9982	.106	.9987	.068	.9992	.031	.9996
11.0	.061	.9993	.043	.9995	.028	.9997	.013	.9998
12.0	0	1.0000	0	1.0000	0	1.0000	0	1.0000

Table 21.10.--(Continued)

T_c = 24 hours

Serial No. :	41		42		43		44	
Q_1/Q_{10} :	0.2		0.3		0.4		0.5	
Time	PSH	PSMC	PSH	PSMC	PSH	PSMC	PSH	PSMC
days	cfs/AQ_{10}	Q/Q_{10}	cfs/AQ_{10}	Q/Q_{10}	cfs/AQ_{10}	Q/Q_{10}	cfs/AQ_{10}	Q/Q_{10}
0	0	0	0	0	0	0	0	0
.3	.132	.0005	.099	.0003	.073	.0002	.058	.0002
.8	1.108	.0113	.831	.0085	.622	.0063	.493	.0050
1.3	1.745	.0452	1.317	.0289	1.029	.0220	.785	.0171
2.0	2.058	.0886	1.641	.0677	1.273	.0521	1.007	.0404
3.0	2.311	.1694	1.940	.1338	1.567	.1041	1.290	.0827
4.0	2.650	.2605	2.432	.2133	2.138	.1711	1.813	.1383
4.6	3.071	.3235	3.016	.2728	2.816	.2248	2.518	.1850
4.9	3.433	.3595	3.652	.3095	3.585	.2599	3.323	.2169
5.0	3.628	.3726	4.052	.3238	4.167	.2742	4.074	.2305
5.1	3.906	.3865	4.674	.3399	5.165	.2914	5.474	.2481
5.2	4.268	.4016	5.529	.3588	6.600	.3131	7.565	.2722
5.3	4.676	.4182	6.517	.3810	8.296	.3406	10.076	.3047
5.4	5.048	.4362	7.417	.4068	9.843	.3741	12.364	.3461
5.5	5.299	.4554	8.000	.4353	10.820	.4122	13.771	.3943
5.6	5.390	.4751	8.180	.4652	11.081	.4526	14.095	.4457
5.7	5.328	.4950	7.984	.4950	10.690	.4928	13.448	.4964
5.8	5.158	.5144	7.544	.5238	9.904	.5308	12.247	.5438
5.9	4.924	.5331	6.981	.5506	8.936	.5656	10.817	.5864
6.0	4.668	.5508	6.387	.5753	7.950	.5967	9.397	.6236
6.2	4.189	.5836	5.336	.6185	6.302	.6491	7.119	.6841
6.4	3.788	.6130	4.505	.6548	5.060	.6909	5.471	.7303
6.6	3.457	.6398	3.864	.6857	4.114	.7246	4.240	.7660
6.9	3.090	.6761	3.227	.7248	3.216	.7648	3.120	.8062
7.2	2.839	.7089	2.785	.7580	2.633	.7970	2.412	.8365
7.6	2.614	.7492	2.396	.7961	2.148	.8320	1.864	.8677
8.0	2.440	.7866	2.115	.8294	1.816	.8612	1.504	.8924
9.0	2.159	.8711	1.728	.8993	1.444	.9202	1.106	.9394
10.0	1.962	.9476	1.528	.9590	1.197	.9691	.913	.9762
10.3	1.660	.9681	1.301	.9750	.984	.9814	.759	.9856
10.8	.670	.9894	.527	.9917	.392	.9938	.304	.9952
11.2	.270	.9960	.212	.9968	.158	.9977	.122	.9982
11.6	.105	.9986	.083	.9989	.061	.9992	.048	.9994
12.5	0	1.0000	0	1.0000	0	1.0000	0	1.0000

Table 21.10.--(Continued)

T_c = 24 hours

Serial No. :	45		46		47		48	
Q_1/Q_{10} :	0.6		0.7		0.8		0.9	
Time	PSH	PSMC	PSH	PSMC	PSH	PSMC	PSH	PSMC
days	cfs/AQ_{10}	Q/Q_{10}	cfs/AQ_{10}	Q/Q_{10}	cfs/AQ_{10}	Q/Q_{10}	cfs/AQ_{10}	Q/Q_{10}
0	0	0	0	0	0	0	0	0
.3	.044	.0002	.029	.0001	.017	.0000	.008	.0000
.8	.370	.0038	.252	.0025	.149	.0015	.071	.0007
1.3	.600	.0130	.430	.0090	.254	.0053	.127	.0026
2.0	.764	.0307	.533	.0216	.362	.0133	.166	.0064
3.0	1.003	.0630	.712	.0442	.477	.0286	.225	.0134
4.0	1.469	.1070	1.072	.0759	.704	.0494	.372	.0240
4.6	2.124	.1457	1.611	.1044	1.155	.0692	.607	.0343
4.9	2.888	.1729	2.306	.1256	1.631	.0843	.864	.0423
5.0	3.800	.1852	3.376	.1361	2.831	.0925	2.199	.0479
5.1	5.632	.2026	5.666	.1527	5.592	.1080	5.458	.0620
5.2	8.450	.2286	9.274	.1802	10.041	.1367	10.806	.0919
5.3	11.874	.2660	13.697	.2225	15.541	.1838	17.458	.1439
5.4	14.982	.3155	17.697	.2803	20.502	.2501	23.431	.2191
5.5	16.845	.3741	20.035	.3498	23.337	.3308	26.765	.3114
5.6	17.205	.4368	20.407	.4243	23.698	.4173	27.079	.4104
5.7	16.242	.4985	19.076	.4970	21.944	.5013	24.844	.5059
5.8	14.568	.5552	16.876	.5632	19.166	.5770	21.438	.5910
5.9	12.633	.6053	14.392	.6207	16.092	.6418	17.740	.6631
6.0	10.755	.6484	12.026	.6694	13.214	.6958	14.330	.7221
6.2	7.851	.7164	8.483	.7441	9.035	.7766	9.512	.8085
6.4	5.804	.7664	6.068	.7973	6.255	.8324	6.370	.8664
6.6	4.290	.8034	4.318	.8353	4.249	.8708	4.110	.9046
6.9	2.972	.8429	2.852	.8742	2.627	.9080	2.352	.9394
7.2	2.168	.8710	1.950	.9003	1.657	.9311	1.334	.9592
7.6	1.581	.8981	1.320	.9238	1.024	.9503	.722	.9736
8.0	1.199	.9185	.925	.9402	.628	.9623	.349	.9813
9.0	.844	.9548	.630	.9676	.392	.9797	.197	.9902
10.0	.678	.9826	.475	.9878	.303	.9922	.140	.9964
10.3	.554	.9895	.390	.9926	.249	.9953	.116	.9978
10.8	.220	.9966	.155	.9976	.099	.9984	.046	.9993
11.2	.088	.9986	.062	.9991	.040	.9994	.018	.9997
11.6	.034	.9995	.024	.9997	.015	.9998	.007	.9999
12.5	0	1.0000	0	1.0000	0	1.0000	0	1.0000

Table 21.10.--(Continued)

T_c = 30 hours

Serial No. :	49		50		51		52	
Q_1/Q_{10} :	0.2		0.3		0.4		0.5	
Time	PSH	PSMC	PSH	PSMC	PSH	PSMC	PSH	PSMC
days	cfs/AQ_{10}	Q/Q_{10}	cfs/AQ_{10}	Q/Q_{10}	cfs/AQ_{10}	Q/Q_{10}	cfs/AQ_{10}	Q/Q_{10}
0	0	0	0	0	0	0	0	0
.4	.150	.0007	.113	.0005	.083	.0004	.067	.0003
.9	.955	.0103	.716	.0077	.538	.0057	.425	.0046
1.5	1.686	.0407	1.281	.0306	.998	.0233	.764	.0181
2.0	1.955	.0747	1.541	.0568	1.195	.0437	.937	.0339
3.0	2.252	.1527	1.872	.1201	1.497	.0932	1.229	.0738
4.0	2.574	.2416	2.316	.1965	2.006	.1567	1.686	.1263
4.6	2.929	.3022	2.814	.2528	2.580	.2068	2.274	.1693
4.9	3.228	.3363	3.306	.2865	3.169	.2383	2.889	.1975
5.1	3.579	.3614	4.022	.3133	4.223	.2651	4.269	.2232
5.2	3.830	.3751	4.582	.3292	5.117	.2823	5.520	.2412
5.3	4.124	.3898	5.258	.3474	6.228	.3032	7.111	.2645
5.4	4.438	.4057	5.994	.3682	7.464	.3285	8.910	.2940
5.5	4.724	.4226	6.662	.3916	8.584	.3581	10.535	.3299
5.6	4.935	.4405	7.144	.4171	9.378	.3913	11.666	.3708
5.7	5.052	.4590	7.397	.4440	9.779	.4266	12.218	.4148
5.8	5.063	.4777	7.391	.4713	9.730	.4626	12.098	.4597
5.9	4.985	.4963	7.182	.4982	9.348	.4978	11.502	.5032
6.0	4.845	.5145	6.841	.5241	8.761	.5312	10.630	.5440
6.2	4.471	.5490	5.976	.5716	7.337	.5907	8.585	.6149
6.4	4.090	.5807	5.149	.6126	6.050	.6400	6.816	.6715
6.6	3.758	.6097	4.479	.6481	5.048	.6808	5.492	.7167
6.9	3.346	.6490	3.706	.6933	3.919	.7301	4.032	.7689
7.2	3.042	.6844	3.159	.7312	3.157	.7689	3.076	.8078
7.6	2.760	.7272	2.658	.7640	2.497	.8103	2.284	.8469
8.0	2.555	.7665	2.313	.8106	2.068	.8438	1.799	.8768
8.6	2.322	.8206	1.957	.8576	1.677	.8849	1.366	.9114
9.2	2.170	.8703	1.738	.8984	1.457	.9194	1.116	.9386
10.0	2.009	.9322	1.566	.9470	1.253	.9594	.951	.9688
10.5	1.530	.9661	1.200	.9734	.915	.9800	.705	.9845
11.0	.702	.9864	.551	.9893	.416	.9920	.321	.9938
11.5	.279	.9949	.219	.9960	.165	.9970	.127	.9977
12.0	.107	.9982	.084	.9986	.063	.9990	.048	.9992
13.0	0	1.0000	0	1.0000	0	1.0000	0	1.0000

Table 21.10.--(Continued)

T_c = 30 hours

Serial No. :	53		54		55		56	
Q_1/Q_{10} :	0.6		0.7		0.8		0.9	
Time	PSH	PSMC	PSH	PSMC	PSH	PSMC	PSH	PSMC
days	cfs/AQ_{10}	Q/Q_{10}	cfs/AQ_{10}	Q/Q_{10}	cfs/AQ_{10}	Q/Q_{10}	cfs/AQ_{10}	Q/Q_{10}
0	0	0	0	0	0	0	0	0
.4	.050	.0002	.033	.0001	.019	.0001	.009	.0000
.9	.320	.0034	.219	.0023	.129	.0013	.062	.0006
1.5	.584	.0137	.416	.0096	.251	.0056	.123	.0027
2.0	.713	.0257	.500	.0180	.332	.0110	.153	.0053
3.0	.949	.0562	.671	.0394	.450	.0253	.211	.0120
4.0	1.355	.0975	.986	.0690	.644	.0449	.336	.0216
4.6	1.898	.1327	1.418	.0948	1.004	.0625	.526	.0308
4.9	2.478	.1566	1.938	.1130	1.373	.0754	.725	.0376
5.1	4.179	.1800	3.972	.1334	3.699	.0923	3.365	.0504
5.2	5.812	.1984	6.010	.1518	6.148	.1104	6.250	.0681
5.3	7.926	.2237	8.685	.1788	9.411	.1390	10.134	.0982
5.4	10.343	.2574	11.774	.2165	13.212	.1806	14.692	.1439
5.5	12.519	.2995	14.539	.2649	16.603	.2355	18.735	.2054
5.6	14.006	.3483	16.397	.3218	18.845	.3007	21.365	.2791
5.7	14.707	.4012	17.246	.3838	19.840	.3718	22.496	.3597
5.8	14.489	.4550	16.905	.4466	19.348	.4439	21.822	.4412
5.9	13.647	.5068	15.784	.5068	17.918	.5125	20.050	.5182
6.0	12.461	.5549	14.256	.5621	16.020	.5749	17.758	.5877
6.2	9.759	.6368	10.855	.6546	11.884	.6777	12.853	.7003
6.4	7.505	.7001	8.116	.7241	8.654	.7527	9.125	.7806
6.6	5.866	.7491	6.195	.7765	6.444	.8080	6.630	.8382
6.9	4.085	.8036	4.141	.8329	4.101	.8654	4.005	.8959
7.2	2.958	.8419	2.860	.8708	2.677	.9019	2.455	.9305
7.6	2.060	.8784	1.856	.9050	1.603	.9328	1.331	.9576
8.0	1.532	.9045	1.292	.9278	1.022	.9517	.757	.9725
8.6	1.082	.9331	.846	.9511	.580	.9691	.349	.9846
9.2	.856	.9541	.639	.9670	.397	.9793	.201	.9900
10.0	.713	.9771	.506	.9838	.319	.9897	.151	.9952
10.5	.517	.9887	.365	.9920	.233	.9949	.109	.9976
11.0	.234	.9955	.165	.9968	.105	.9980	.049	.9991
11.5	.093	.9983	.065	.9988	.042	.9992	.019	.9996
12.0	.035	.9994	.025	.9996	.016	.9997	.007	.9999
13.0	0	1.0000	0	1.0000	0	1.0000	0	1.0000

Table 21.10.--(Continued)

T_c = 36 hours

Serial No. :	57		58		59		60	
Q_1/Q_{10} :	0.2		0.3		0.4		0.5	
Time	PSH	PSMC	PSH	PSMC	PSH	PSMC	PSH	PSMC
days	cfs/AQ_{10}	Q/Q_{10}	cfs/AQ_{10}	Q/Q_{10}	cfs/AQ_{10}	Q/Q_{10}	cfs/AQ_{10}	Q/Q_{10}
0	0	0	0	0	0	0	0	0
.5	.163	.0009	.122	.0007	.091	.0005	.072	.0004
1.2	1.130	.0170	.848	.0127	.648	.0095	.504	.0075
2.0	1.817	.0623	1.418	.0473	1.101	.0363	.857	.0281
3.0	2.177	.1368	1.794	.1072	1.424	.0830	1.165	.0656
4.0	2.498	.2231	2.211	.1805	1.889	.1433	1.576	.1152
4.8	2.964	.3032	2.884	.2544	2.666	.2088	2.366	.1714
5.0	3.176	.3259	3.250	.2770	3.133	.2301	2.892	.1906
5.1	3.331	.3380	3.565	.2896	3.598	.2425	3.506	.2024
5.2	3.521	.3506	3.965	.3036	4.212	.2569	4.339	.2169
5.3	3.742	.3641	4.451	.3192	4.982	.2739	5.411	.2349
5.4	3.987	.3784	5.002	.3366	5.874	.2940	6.673	.2572
5.5	4.238	.3937	5.574	.3562	6.814	.3174	8.017	.2843
5.6	4.467	.4098	6.095	.3778	7.670	.3442	9.240	.3162
5.7	4.644	.4267	6.492	.4011	8.311	.3737	10.141	.3519
5.8	4.760	.4441	6.741	.4256	8.704	.4051	10.682	.3904
5.9	4.806	.4618	6.826	.4507	8.824	.4375	10.825	.4301
6.0	4.784	.4796	6.757	.4758	8.686	.4698	10.598	.4696
6.1	4.708	.4972	6.567	.5005	8.354	.5013	10.099	.5078
6.2	4.593	.5144	6.293	.5243	7.898	.5314	9.435	.5439
6.4	4.296	.5474	5.623	.5684	6.815	.5858	7.902	.6080
6.6	3.984	.5781	4.960	.6076	5.787	.6323	6.494	.6610
6.8	3.704	.6066	4.403	.6422	4.956	.6719	5.399	.7048
7.1	3.348	.6457	3.736	.6872	3.989	.7212	4.151	.7573
7.5	2.989	.6925	3.078	.7373	3.072	.7729	2.997	.8095
8.0	2.680	.7449	2.536	.7890	2.366	.8227	2.159	.8565
8.6	2.414	.8014	2.108	.8402	1.861	.8690	1.583	.8973
9.2	2.230	.8529	1.837	.8838	1.568	.9068	1.248	.9285
10.0	2.052	.9163	1.610	.9344	1.308	.9490	.994	.9609
10.5	1.710	.9519	1.343	.9623	1.045	.9712	.803	.9778
11.0	.978	.9769	.768	.9819	.587	.9862	.453	.9894
11.6	.391	.9912	.307	.9932	.234	.9948	.180	.9960
12.5	.092	.9982	.072	.9986	.055	.9990	.042	9992
14.0	0	1.0000	0	1.0000	0	1.0000	0	1.0000

Table 21.10.--(Continued)

T_c = 36 hours

Serial No. :	61		62		63		64	
Q_1/Q_{10} :	0.6		0.7		0.8		0.9	
Time	PSH	PSMC	PSH	PSMC	PSH	PSMC	PSH	PSMC
days	cfs/AQ_{10}	Q/Q_{10}	cfs/AQ_{10}	Q/Q_{10}	cfs/AQ_{10}	Q/Q_{10}	cfs/AQ_{10}	Q/Q_{10}
0	0	0	0	0	0	0	0	0
.5	.054	.0003	.036	.0002	.021	.0001	.010	.0000
1.2	.382	.0056	.266	.0039	.156	.0023	.077	.0011
2.0	.653	.0214	.460	.0150	.298	.0091	.140	.0044
3.0	.895	.0498	.631	.0349	.422	.0224	.198	.0106
4.0	1.258	.0887	.911	.0627	.596	.0407	.306	.0195
4.8	1.986	.1347	1.504	.0965	1.059	.0638	.555	.0315
5.0	2.536	.1510	2.062	.1092	1.574	.0730	1.014	.0367
5.1	3.301	.1618	2.986	.1185	2.645	.0808	2.244	.0427
5.2	4.360	.1759	4.282	.1319	4.170	.0933	4.011	.0542
5.3	5.752	.1946	6.014	.1509	6.245	.1125	6.449	.0735
5.4	7.411	.2189	8.096	.1769	8.766	.1401	9.430	.1027
5.5	9.193	.2495	10.347	.2109	11.507	.1775	12.688	.1434
5.6	10.811	.2864	12.385	.2528	13.984	.2244	15.620	.1956
5.7	11.982	.3284	13.839	.3011	15.724	.2792	17.650	.2568
5.8	12.675	.3739	14.683	.3537	16.719	.3389	18.791	.3239
5.9	12.833	.4209	14.846	.4081	16.878	.4008	18.923	.3934
6.0	12.498	.4676	14.387	.4620	16.277	.4619	18.170	.4617
6.1	11.814	.5125	13.499	.5134	15.167	.5198	16.818	.5261
6.2	10.927	.5544	12.374	.5611	13.786	.5732	15.168	.5850
6.4	8.922	.6277	9.876	.6432	10.772	.6638	11.616	.6838
6.6	7.130	.6868	7.709	.7079	8.215	.7335	8.662	.7581
6.8	5.776	.7342	6.123	.7586	6.390	.7870	6.602	.8140
7.1	4.260	.7893	4.373	.8161	4.400	.8459	4.379	.8738
7.5	2.893	.8412	2.805	.8680	2.648	.8966	2.461	.9228
8.0	1.947	.8852	1.756	.9092	1.528	.9343	1.295	.9564
8.6	1.323	.9206	1.105	.9400	.855	.9596	.632	.9767
9.2	.991	.9460	.779	.9607	.540	.9749	.340	.9873
10.0	.752	.9711	.542	.9794	.340	.9870	.164	.9938
10.5	.594	.9837	.422	.9884	.268	.9927	.126	.9966
11.0	.332	.9922	.236	.9945	.150	.9965	.070	.9984
11.6	.132	.9971	.094	.9979	.059	.9987	.028	.9994
12.5	.031	.9994	.022	.9996	.014	.9997	.006	.9999
14.0	0	1.0000	0	1.0000	0	1.0000	0	1.0000

Table 21.10.--(Continued)

T_c = 42 hours

Serial No. :	65		66		67		68	
Q_1/Q_{10} :	0.2		0.3		0.4		0.5	
Time	PSH	PSMC	PSH	PSMC	PSH	PSMC	PSH	PSMC
days	cfs/AQ_{10}	Q/Q_{10}	cfs/AQ_{10}	Q/Q_{10}	cfs/AQ_{10}	Q/Q_{10}	cfs/AQ_{10}	Q/Q_{10}
0	0	0	0	0	0	0	0	0
.6	.174	.0011	.131	.0008	.097	.0006	.078	.0005
1.2	.892	.0123	.670	.0092	.509	.0069	.398	.0054
2.0	1.666	.0516	1.290	.0391	1.001	.0300	.777	.0232
3.0	2.097	.1220	1.714	.0952	1.354	.0737	1.101	.0580
4.0	2.428	.2056	2.120	.1655	1.789	.1309	1.484	.1050
4.8	2.846	.2829	2.711	.2358	2.466	.1922	2.162	.1572
5.0	3.026	.3046	3.007	.2568	2.834	.2117	2.570	.1744
5.2	3.301	.3280	3.550	.2809	3.630	.2353	3.610	.1969
5.4	3.669	.3537	4.330	.3099	4.841	.2664	5.268	.2294
5.5	3.875	.3677	4.784	.3268	5.564	.2856	6.278	.2507
5.6	4.082	.3824	5.245	.3453	6.306	.3075	7.325	.2757
5.7	4.272	.3979	5.667	.3654	6.989	.3320	8.287	.3045
5.8	4.425	.4139	6.004	.3870	7.524	.3588	9.031	.3364
5.9	4.536	.4305	6.241	.4096	7.898	.3872	9.546	.3707
6.0	4.597	.4474	6.366	.4329	8.086	.4167	9.795	.4063
6.1	4.606	.4644	6.370	.4564	8.075	.4465	9.756	.4423
6.2	4.569	.4814	6.272	.4798	7.899	.4759	9.484	.4778
6.3	4.497	.4982	6.098	.5026	7.608	.5045	9.058	.5119
6.4	4.399	.5146	5.872	.5247	7.239	.5319	8.531	.5444
6.6	4.155	.5463	5.338	.5662	6.391	.5822	7.346	.6029
6.8	3.895	.5761	4.795	.6036	5.554	.6262	6.206	.6528
7.0	3.653	.6040	4.317	.6372	4.840	.6645	5.262	.6949
7.3	3.343	.6428	3.734	.6818	4.001	.7133	4.182	.7469
7.6	3.088	.6784	3.266	.7205	3.348	.7538	3.359	.7884
8.0	2.820	.7220	2.784	.7650	2.700	.7981	2.568	.8317
8.5	2.565	.7718	2.355	.8122	2.165	.8427	1.943	.8729
9.2	2.310	.8346	1.961	.8676	1.713	.8922	1.420	.9156
10.0	2.110	.9000	1.683	.9212	1.397	.9379	1.084	.9522
10.5	1.840	.9370	1.451	.9504	1.151	.9616	.884	.9704
11.2	.967	.9737	.762	.9794	.588	.9842	.454	.9878
12.0	.334	.9915	.263	.9933	.202	.9949	.156	.9961
12.8	.110	.9975	.086	.9980	.066	.9985	.051	.9988
14.5	0	1.0000	0	1.0000	0	1.0000	0	1.0000

Table 21.10.--(Continued)

T_c = 42 hours

Serial No. :	69		70		71		72	
Q_1/Q_{10} :	0.6		0.7		0.8		0.9	
Time	PSH	PSMC	PSH	PSMC	PSH	PSMC	PSH	PSMC
days	cfs/AQ_{10}	Q/Q_{10}	cfs/AQ_{10}	Q/Q_{10}	cfs/AQ_{10}	Q/Q_{10}	cfs/AQ_{10}	Q/Q_{10}
0	0	0	0	0	0	0	0	0
.6	.058	.0004	.039	.0002	.023	.0001	.010	.0001
1.2	.301	.0041	.209	.0028	.122	.0016	.060	.0008
2.0	.592	.0176	.418	.0123	.266	.0074	.126	.0036
3.0	.844	.0441	.593	.0309	.396	.0197	.186	.0093
4.0	1.178	.0807	.850	.0569	.558	.0370	.282	.0177
4.8	1.798	.1230	1.347	.0879	.939	.0579	.490	.0285
5.0	2.216	.1376	1.759	.0990	1.313	.0658	.810	.0328
5.2	3.499	.1582	3.293	.1172	3.079	.0814	2.818	.0455
5.4	5.621	.1914	5.903	.1505	6.173	.1148	6.419	.0788
5.5	6.936	.2146	7.540	.1752	8.138	.1412	8.728	.1066
5.6	8.308	.2426	9.258	.2062	10.212	.1749	11.174	.1432
5.7	9.567	.2755	10.832	.2431	12.111	.2159	13.409	.1884
5.8	10.527	.3125	12.016	.2852	13.520	.2631	15.045	.2406
5.9	11.188	.3525	12.824	.3309	14.476	.3146	16.148	.2980
6.0	11.495	.3943	13.187	.3787	14.890	.3686	16.606	.3582
6.1	11.418	.4364	13.063	.4270	14.708	.4230	16.354	.4188
6.2	11.040	.4778	12.566	.4742	14.078	.4759	15.577	.4774
6.3	10.467	.5174	11.836	.5191	13.180	.5260	14.500	.5327
6.4	9.774	.5547	10.968	.5610	12.127	.5726	13.251	.5837
6.6	8.239	.6211	9.079	.6349	9.864	.6535	10.602	.6715
6.8	6.794	.6763	7.338	.6951	7.814	.7183	8.237	.7404
7.0	5.622	.7218	5.958	.7438	6.217	.7696	6.425	.7940
7.3	4.315	.7764	4.449	.8010	4.506	.8284	4.519	.8540
7.6	3.340	.8185	3.331	.8437	3.257	.8710	3.151	.8960
8.0	2.424	.8604	2.296	.8844	2.123	.9097	1.937	.9324
8.5	1.726	.8982	1.540	.9192	1.320	.9409	1.114	.9599
9.2	1.174	.9348	.973	.9507	.742	.9665	.545	.9802
10.0	.844	.9642	.637	.9742	.428	.9835	.250	.9919
10.5	.662	.9782	.475	.9844	.301	.9901	.144	.9953
11.2	.335	.9910	.239	.9936	.152	.9960	.072	.9981
12.0	.115	.9971	.082	.9980	.052	.9987	.024	.9994
12.8	.037	.9992	.026	.9994	.017	.9996	.008	.9998
14.5	0	1.0000	0	1.0000	0	1.0000	0	1.0000

Table 21.10.--(Continued)

T_c = 48 hours

Serial No. :	73		74		75		76	
Q_1/Q_{10} :	0.2		0.3		0.4		0.5	
Time	PSH	PSMC	PSH	PSMC	PSH	PSMC	PSH	PSMC
days	cfs/AQ_{10}	Q/Q_{10}	cfs/AQ_{10}	Q/Q_{10}	cfs/AQ_{10}	Q/Q_{10}	cfs/AQ_{10}	Q/Q_{10}
0	0	0	0	0	0	0	0	0
.6	.120	.0008	.090	.0006	.067	.0004	.054	.0003
1.3	.811	.0118	.610	.0088	.464	.0066	.362	.0052
2.0	1.500	.0425	1.155	.0321	.895	.0246	.694	.0191
3.0	2.001	.1083	1.624	.0842	1.278	.0651	1.033	.0512
4.0	2.350	.1888	2.027	.1514	1.692	.1193	1.398	.0955
4.8	2.733	.2635	2.557	.2182	2.291	.1770	1.988	.1442
5.0	2.888	.2843	2.803	.2380	2.595	.1949	2.320	.1599
5.2	3.116	.3065	3.230	.2602	3.203	.2162	3.099	.1797
5.4	3.413	.3306	3.831	.2862	4.107	.2430	4.305	.2068
5.5	3.585	.3436	4.194	.3010	4.670	.2592	5.077	.2241
5.6	3.763	.3572	4.576	.3172	5.272	.2775	5.909	.2443
5.7	3.939	.3714	4.958	.3348	5.877	.2981	6.752	.2677
5.8	4.100	.3863	5.310	.3538	6.437	.3208	7.532	.2940
5.9	4.235	.4017	5.600	.3740	6.892	.3454	8.160	.3229
6.0	4.335	.4176	5.812	.3951	7.221	.3714	8.606	.3538
6.1	4.400	.4338	5.943	.4168	7.419	.3985	8.868	.3860
6.2	4.428	.4501	5.992	.4388	7.486	.4260	8.946	.4189
6.3	4.420	.4665	5.962	.4610	7.427	.4535	8.848	.4517
6.4	4.379	.4828	5.863	.4828	7.257	.4806	8.596	.4839
6.6	4.228	.5146	5.521	.5249	6.702	.5322	7.805	.5444
6.8	4.025	.5452	5.084	.5642	6.017	.5791	6.860	.5986
7.0	3.804	.5742	4.630	.6000	5.322	.6209	5.917	.6456
7.3	3.499	.6147	4.037	.6480	4.446	.6748	4.766	.7044
7.6	3.240	.6521	3.556	.6900	3.765	.7202	3.899	.7521
8.0	2.956	.6979	3.037	.7386	3.053	.7702	3.014	.8028
8.5	2.677	.7499	2.552	.7900	2.425	.8203	2.259	.8508
9.2	2.393	.8153	2.097	.8497	1.881	.8753	1.621	.9002
10.0	2.171	.8827	1.775	.9065	1.512	.9250	1.213	.9413
10.5	1.944	.9212	1.558	.9376	1.273	.9508	1.003	.9618
11.2	1.189	.9625	.942	.9704	.737	.9770	.570	.9823
12.0	.478	.9858	.377	.9889	.293	.9914	.226	.9934
13.0	.142	.9962	.112	.9970	.086	.9977	.066	.9982
15.0	0	1.0000	0	1.0000	0	1.0000	0	1.0000

Table 21.10.--(Continued)

T_c = 48 hours

Serial No. :	77		78		79		80	
Q_1/Q_{10} :	0.6		0.7		0.8		0.9	
Time	PSH	PSMC	PSH	PSMC	PSH	PSMC	PSH	PSMC
days	cfs/AQ_{10}	Q/Q_{10}	cfs/AQ_{10}	Q/Q_{10}	cfs/AQ_{10}	Q/Q_{10}	cfs/AQ_{10}	Q/Q_{10}
0	0	0	0	0	0	0	0	0
.6	.040	.0002	.027	.0002	.016	.0001	.007	.0000
1.3	.274	.0039	.190	.0027	.112	.0016	.055	.0008
2.0	.528	.0145	.373	.0101	.234	.0061	.112	.0029
3.0	.790	.0389	.555	.0272	.370	.1727	.173	.0082
4.0	1.104	.0732	.793	.0516	.522	.0334	.261	.0160
4.8	1.639	.1125	1.218	.0802	.840	.0526	.437	.0258
5.0	1.977	.1256	1.545	.0901	1.138	.0596	.689	.0296
5.2	2.918	.1434	2.651	.1052	2.395	.0722	2.098	.0394
5.4	4.429	.1702	4.476	.1311	4.524	.0972	4.538	.0632
5.5	5.419	.1882	5.696	.1498	5.972	.1166	6.226	.0830
5.6	6.493	.2102	7.024	.1732	7.558	.1414	8.080	.1093
5.7	7.586	.2362	8.382	.2016	9.183	.1722	9.984	.1426
5.8	8.600	.2660	9.640	.2348	10.691	.2088	11.750	.1825
5.9	9.405	.2991	10.630	.2721	11.865	.2503	13.110	.2282
6.0	9.970	.3348	11.315	.3125	12.668	.2954	14.028	.2781
6.1	10.296	.3722	11.702	.3549	13.112	.3429	14.526	.3306
6.2	10.381	.4102	11.791	.3981	13.198	.3913	14.603	.3842
6.3	10.238	.4482	11.598	.4412	12.948	.4394	14.287	.4373
6.4	9.898	.4853	11.163	.4831	12.407	.4860	13.633	.4887
6.6	8.858	.5545	9.866	.5606	10.836	.5717	11.770	.5823
6.8	7.644	.6154	8.386	.6279	9.075	.6450	9.720	.6613
7.0	6.452	.6671	6.954	.6841	7.390	.7053	7.779	.7253
7.3	5.034	.7302	5.290	.7512	5.473	.7757	5.612	.7984
7.6	3.995	.7799	4.095	.8029	4.128	.8284	4.125	.8519
8.0	2.954	.8308	2.908	.8540	2.810	.8789	2.693	.9014
8.5	2.092	.8764	1.951	.8977	1.772	.9200	1.599	.9396
9.2	1.397	.9206	1.213	.9376	.998	.9546	.812	.9695
10.0	.985	.9550	.791	.9663	.587	.9770	.416	.9866
10.5	.784	.9713	.599	.9791	.422	.9863	.265	.9928
11.2	.425	.9869	.305	.9906	.194	.9941	.093	.9972
12.0	.168	.9951	.120	.9965	.076	.9978	.036	.9990
13.0	.049	.9987	.035	.9991	.022	.9994	.010	.9997
15.0	0	1.0000	0	1.0000	0	1.0000	0	1.0000

Table 21.10.--(Continued)

T_c = 54 hours

Serial No. :	81		82		83		84	
Q_1/Q_{10} :	0.2		0.3		0.4		0.5	
Time	PSH	PSMC	PSH	PSMC	PSH	PSMC	PSH	PSMC
days	cfs/AQ_{10}	Q/Q_{10}	cfs/AQ_{10}	Q/Q_{10}	cfs/AQ_{10}	Q/Q_{10}	cfs/AQ_{10}	Q/Q_{10}
0	0	0	0	0	0	0	0	0
.6	.087	.0005	.065	.0004	.048	.0003	.039	.0002
1.3	.640	.0089	.481	.0067	.366	.0050	.286	.0039
2.0	1.331	.0349	1.020	.0263	.790	.0201	.612	.0156
3.0	1.897	.0957	1.529	.0742	1.199	.0573	.965	.0450
4.0	2.269	.1730	1.937	.1381	1.601	.1085	1.319	.0867
4.8	2.631	.2451	2.422	.2018	2.141	.1629	1.841	.1323
5.0	2.768	.2650	2.633	.2204	2.398	.1795	2.118	.1468
5.2	2.961	.2862	2.977	.2411	2.876	.1989	2.717	.1644
5.4	3.210	.3090	3.462	.2648	3.589	.2226	3.652	.1877
5.6	3.504	.3338	4.063	.2926	4.502	.2524	4.883	.2191
5.8	3.811	.3609	4.713	.3250	5.516	.2894	6.275	.2602
6.0	4.070	.3901	5.261	.3620	6.370	.3334	7.446	.3110
6.1	4.164	.4054	5.454	.3181	6.666	.3574	7.844	.3392
6.2	4.231	.4209	5.591	.4022	6.873	.3824	8.121	.3686
6.3	4.271	.4366	5.667	.4230	6.986	.4080	8.269	.3989
6.4	4.278	.4524	5.672	.4439	6.982	.4337	8.246	.4293
6.5	4.260	.4682	5.621	.4648	6.891	.4593	8.108	.4594
6.6	4.219	.4839	5.524	.4854	6.732	.4845	7.876	.4889
6.8	4.085	.5147	5.230	.5252	6.262	.5325	7.216	.5446
7.0	3.912	.5443	4.866	.5625	5.697	.5766	6.443	.5950
7.3	3.630	.5862	4.298	.6132	4.840	.6348	5.294	.6597
7.6	3.373	.6250	3.808	.6581	4.130	.6843	4.372	.7129
8.0	3.085	.6727	3.278	.7104	3.392	.7396	3.445	.7703
8.5	2.790	.7270	2.754	.7659	2.695	.7955	2.591	.8256
9.0	2.560	.7764	2.371	.8131	2.218	.8405	2.025	.8678
9.5	2.381	.8220	2.086	.8541	1.880	.8782	1.627	.9013
10.0	2.237	.8647	1.874	.8906	1.632	.9105	1.349	.9286
10.6	1.969	.9119	1.603	.9294	1.335	.9435	1.075	.9554
11.2	1.389	.9496	1.119	.9600	.902	.9684	.718	.9754
12.0	.635	.9787	.504	.9832	.397	.9869	.308	.9899
13.0	.218	.9932	.172	.9946	.134	.9958	.104	.9968
14.0	.071	.9980	.056	.9985	.043	.9988	.033	.9991
16.0	0	1.0000	0	1.0000	0	1.0000	0	1.0000

Table 21.10.--(Continued)

T_c = 54 hours

Serial No. :	85		86		87		88	
Q_1/Q_{10} :	0.6		0.7		0.8		0.9	
Time	PSH	PSMC	PSH	PSMC	PSH	PSMC	PSH	PSMC
days	cfs/AQ_{10}	Q/Q_{10}	cfs/AQ_{10}	Q/Q_{10}	cfs/AQ_{10}	Q/Q_{10}	cfs/AQ_{10}	Q/Q_{10}
0	0	0	0	0	0	0	0	0
.6	.029	.0002	.019	.0001	.011	.0001	.005	.0000
1.3	.216	.0030	.150	.0020	.088	.0012	.043	.0006
2.0	.466	.0118	.328	.0082	.204	.0049	.098	.0024
3.0	.736	.0342	.517	.0239	.343	.0151	.160	.0072
4.0	1.036	.0664	.742	.0468	.490	.0302	.242	.0144
4.8	1.506	.1029	1.112	.0732	.760	.0479	.394	.0234
5.0	1.786	.1149	1.378	.0822	1.003	.0542	.597	.0268
5.2	2.494	.1304	2.194	.0951	1.916	.0646	1.604	.0345
5.4	3.650	.1528	3.575	.1160	3.512	.0842	3.419	.0525
5.6	5.204	.1853	5.464	.1491	5.730	.1180	5.975	.0868
5.8	6.992	.2302	7.667	.1974	8.350	.1698	9.207	.1419
6.0	8.492	.2876	9.507	.2611	10.530	.2398	11.554	.2182
6.1	8.993	.3198	10.113	.2972	11.237	.2798	12.360	.2622
6.2	9.340	.3535	10.531	.3352	11.722	.3220	12.912	.3087
6.3	9.523	.3883	10.748	.3744	11.972	.3656	13.191	.3567
6.4	9.476	.4233	10.674	.4138	11.860	.4094	13.035	.4049
6.5	9.286	.4578	10.427	.4526	11.551	.4525	12.656	.4521
6.6	8.978	.4914	10.041	.4903	11.079	.4941	12.092	.4976
6.8	8.120	.5545	8.982	.5604	9.805	.5711	10.593	.5812
7.0	7.135	.6107	7.792	.6222	8.398	.6381	8.962	.6531
7.3	5.694	.6813	6.072	.6984	6.386	.7192	6.657	.7386
7.6	4.572	.7377	4.765	.7578	4.894	.7809	4.985	.8022
8.0	3.472	.7967	3.505	.8183	3.484	.8421	3.439	.8637
8.5	2.478	.8511	2.385	.8721	2.250	.8944	2.113	.9142
9.0	1.847	.8904	1.701	.9091	1.521	.9284	1.358	.9453
9.5	1.417	.9203	1.245	.9360	1.044	.9518	.871	.9656
10.0	1.129	.9436	.944	.9560	.744	.9680	.575	.9787
10.6	.866	.9656	.689	.9739	.517	.9818	.366	.9888
11.2	.561	.9814	.432	.9863	.310	.9909	.201	.9951
12.0	.230	.9925	.166	.9946	.105	.9966	.050	.9984
13.0	.077	.9976	.055	.9983	.035	.9989	.017	.9995
14.0	.025	.9993	.018	.9995	.011	.9997	.005	.9999
16.0	0	1.0000	0	1.0000	0	1.0000	0	1.0000

Table 21.10.--(Continued)

T_c = 60 hours

Serial No. :	89		90		91		92	
Q_1/Q_{10} :	0.2		0.3		0.4		0.5	
Time	PSH	PSMC	PSH	PSMC	PSH	PSMC	PSH	PSMC
days	cfs/AQ_{10}	Q/Q_{10}	cfs/AQ_{10}	Q/Q_{10}	cfs/AQ_{10}	Q/Q_{10}	cfs/AQ_{10}	Q/Q_{10}
0	0	0	0	0	0	0	0	0
.6	.065	.0004	.048	.0003	.036	.0002	.029	.0002
1.3	.506	.0068	.380	.0051	.288	.0038	.226	.0030
2.0	1.164	.0286	.890	.0216	.687	.0164	.533	.0128
3.0	1.785	.0844	1.430	.0652	1.119	.0503	.896	.0394
4.0	2.184	.1580	1.848	.1258	1.515	.0986	1.244	.0786
4.8	2.534	.2276	2.298	.1864	2.008	.1498	1.713	.1214
5.0	2.658	.2468	2.483	.2041	2.229	.1654	1.949	.1348
5.2	2.824	.2670	2.769	.2234	2.617	.1832	2.423	.1508
5.4	3.038	.2887	3.168	.2453	3.189	.2045	3.160	.1712
5.6	3.290	.3121	3.665	.2705	3.929	.2307	4.142	.1980
5.8	3.562	.3374	4.221	.2996	4.777	.2628	5.287	.2327
6.0	3.819	.3648	4.757	.3329	5.606	.3012	6.420	.2760
6.2	4.012	.3938	5.150	.3695	6.204	.3449	7.221	.3264
6.3	4.080	.4088	5.286	.3888	6.411	.3681	7.499	.3535
6.4	4.122	.4239	5.366	.4085	6.526	.3920	7.645	.3815
6.5	4.142	.4392	5.399	.4284	6.570	.4162	7.694	.4097
6.6	4.140	.4546	5.388	.4483	6.544	.4404	7.650	.4380
6.8	4.077	.4850	5.237	.4877	6.293	.4878	7.284	.4932
7.0	3.959	.5148	4.981	.5254	5.890	.5328	6.723	.5449
7.2	3.809	.5435	4.671	.5611	5.415	.5745	6.078	.5921
7.4	3.645	.5711	4.339	.5944	4.912	.6126	5.402	.6344
7.7	3.408	.6102	3.881	.6400	4.242	.6632	4.525	.6892
8.0	3.199	.6469	3.493	.6808	3.698	.7071	3.838	.7353
8.5	2.901	.7032	2.958	.7402	2.976	.7684	2.946	.7974
9.0	2.656	.7546	2.535	.7908	2.430	.8180	2.283	.8453
9.5	2.463	.8019	2.224	.8346	2.055	.8591	1.839	.8830
10.0	2.306	.8460	1.985	.8734	1.769	.8943	1.510	.9138
10.6	2.059	.8948	1.708	.9145	1.456	.9301	1.199	.9437
11.2	1.561	.9354	1.277	.9479	1.055	.9581	.859	.9666
12.0	.806	.9699	.653	.9760	.529	.9810	.424	.9852
13.0	.308	.9891	.245	.9914	.193	.9933	.150	.9948
14.0	.114	.9963	.090	.9971	.070	.9978	.054	.9983
16.0	0	1.0000	0	1.0000	0	1.0000	0	1.0000

Table 21.10.--(Continued)

T_c = 60 hours

Serial No. :	93		94		95		96	
Q_1/Q_{10} :	0.6		0.7		0.8		0.9	
Time	PSH	PSMC	PSH	PSMC	PSH	PSMC	PSH	PSMC
days	cfs/AQ_{10}	Q/Q_{10}	cfs/AQ_{10}	Q/Q_{10}	cfs/AQ_{10}	Q/Q_{10}	cfs/AQ_{10}	Q/Q_{10}
0	0	0	0	0	0	0	0	0
.6	.022	.0001	.014	.0001	.008	.0000	.004	.0000
1.3	.170	.0023	.118	.0016	.069	.0009	.034	.0004
2.0	.405	.0097	.285	.0067	.176	.0040	.084	.0019
3.0	.683	.0299	.480	.0210	.317	.0132	.148	.0063
4.0	.973	.0602	.695	.0423	.459	.0273	.224	.0130
4.8	1.392	.0942	1.022	.0669	.694	.0437	.357	.0212
5.0	1.627	.1052	1.244	.0751	.895	.0494	.525	.0243
5.2	2.178	.1190	1.865	.0863	1.580	.0582	1.268	.0306
5.4	3.074	.1382	2.921	.1038	2.786	.0741	2.626	.0447
5.6	4.299	.1652	4.394	.1305	4.503	.1007	4.590	.0709
5.8	5.750	.2022	6.161	.1692	6.582	.1413	6.989	.1133
6.0	7.197	.2499	7.936	.2212	8.687	.1976	9.432	.1738
6.2	8.204	.3069	9.151	.2844	10.102	.2670	11.047	.2494
6.3	8.555	.3377	9.578	.3188	10.602	.3050	11.622	.2911
6.4	8.729	.3696	9.779	.3545	10.824	.3445	11.860	.3342
6.5	8.782	.4018	9.835	.3906	10.878	.3844	11.908	.3780
6.6	8.718	.4340	9.750	.4266	10.768	.4242	11.770	.4215
6.8	8.230	.4966	9.138	.4963	10.016	.5008	10.868	.5049
7.0	7.507	.5546	8.255	.5604	8.961	.5707	9.633	.5804
7.2	6.690	.6069	7.272	.6176	7.804	.6324	8.298	.6463
7.4	5.840	.6531	6.254	.6673	6.610	.6854	6.924	.7022
7.7	4.765	.7116	4.994	.7293	5.162	.7502	5.294	.7694
8.0	3.946	.7595	4.055	.7791	4.109	.8012	4.136	.8212
8.5	2.903	.8220	2.874	.8422	2.803	.8640	2.723	.8834
9.0	2.142	.8681	2.027	.8868	1.876	.9065	1.735	.9237
9.5	1.657	.9028	1.510	.9190	1.332	.9356	1.178	.9501
10.0	1.306	.9299	1.136	.9432	.945	.9564	.784	.9680
10.6	.994	.9552	.821	.9647	.647	.9738	.495	.9819
11.2	.693	.9739	.558	.9798	.428	.9855	.312	.9906
12.0	.334	.9887	.261	.9917	.191	.9945	.128	.9970
13.0	.112	.9962	.081	.9972	.051	.9982	.025	.9992
14.0	.040	.9987	.029	.9991	.018	.9994	.009	.9997
16.0	0	1.0000	0	1.0000	0	1.0000	0	1.0000

Table 21.10.--(Continued)

T_c = 66 hours

Serial No. :	97		98		99		100	
Q_1/Q_{10} :	0.2		0.3		0.4		0.5	
Time	PSH	PSMC	PSH	PSMC	PSH	PSMC	PSH	PSMC
days	cfs/AQ_{10}	Q/Q_{10}	cfs/AQ_{10}	Q/Q_{10}	cfs/AQ_{10}	Q/Q_{10}	cfs/AQ_{10}	Q/Q_{10}
0	0	0	0	0	0	0	0	0
.6	.050	.0003	.037	.0002	.028	.0002	.022	.0001
1.3	.401	.0053	.302	.0040	.229	.0030	.179	.0023
2.0	1.008	.0234	.769	.0177	.593	.0135	.460	.0105
3.0	1.670	.0741	1.330	.0572	1.039	.0441	.828	.0345
4.0	2.095	.1441	1.758	.1143	1.431	.0894	1.171	.0712
4.8	2.438	.2110	2.184	.1720	1.887	.1378	1.599	.1113
5.0	2.553	.2294	2.349	.1888	2.082	.1524	1.803	.1238
5.2	2.701	.2488	2.593	.2070	2.406	.1688	2.193	.1384
5.4	2.887	.2695	2.926	.2273	2.872	.1882	2.781	.1567
5.6	3.106	.2917	3.342	.2505	3.478	.2116	3.572	.1800
5.8	3.348	.3156	3.819	.2769	4.191	.2399	4.520	.2098
6.0	3.589	.3412	4.308	.3070	4.934	.2736	5.521	.2468
6.2	3.796	.3686	4.726	.3404	5.570	.3124	6.377	.2909
6.4	3.942	.3972	5.015	.3765	6.003	.3552	6.952	.3401
6.5	3.987	.4119	5.102	.3952	6.129	.3776	7.114	.3661
6.6	4.015	.4267	5.154	.4141	6.205	.4004	7.211	.3925
6.7	4.023	.4416	5.160	.4332	6.204	.4233	7.197	.4190
6.8	4.014	.4565	5.133	.4522	6.157	.4461	7.125	.4454
7.0	3.951	.4860	4.988	.4897	5.921	.4907	6.787	.4968
7.2	3.846	.5148	4.764	.5257	5.571	.5332	6.305	.5451
7.4	3.716	.5428	4.497	.5600	5.165	.5728	5.756	.5896
7.7	3.502	.5829	4.070	.6074	4.528	.6264	4.910	.6485
8.0	3.297	.6206	3.679	.6504	3.964	.6733	4.181	.6987
8.5	3.001	.6788	3.142	.7132	3.230	.7394	3.266	.7669
9.0	2.752	.7320	2.707	.7671	2.661	.7935	2.570	.8204
9.5	2.545	.7809	2.362	.8138	2.228	.8384	2.048	.8627
10.0	2.379	.8264	2.103	.8549	1.917	.8765	1.688	.8970
10.6	2.143	.8769	1.816	.8985	1.585	.9153	1.339	.9304
11.2	1.708	.9200	1.418	.9346	1.196	.9462	.992	.9562
12.0	.978	.9596	.806	.9672	.669	.9734	.552	.9787
13.0	.410	.9839	.332	.9872	.269	.9898	.215	.9921
14.0	.166	.9939	.132	.9952	.104	.9962	.081	.9971
17.0	0	1.0000	0	1.0000	0	1.0000	0	1.0000

Table 21.10.--(Continued)

T_c = 66 hours

Serial No. :	101		102		103		104	
Q_1/Q_{10} :	0.6		0.7		0.8		0.9	
Time	PSH	PSMC	PSH	PSMC	PSH	PSMC	PSH	PSMC
days	cfs/AQ_{10}	Q/Q_{10}	cfs/AQ_{10}	Q/Q_{10}	cfs/AQ_{10}	Q/Q_{10}	cfs/AQ_{10}	Q/Q_{10}
0	0	0	0	0	0	0	0	0
.6	.016	.0001	.011	.0001	.006	.0000	.003	.0000
1.3	.135	.0018	.094	.0012	.055	.0007	.027	.0003
2.0	.350	.0079	.246	.0055	.150	.0033	.072	.0016
3.0	.631	.0262	.443	.0183	.291	.0115	.136	.0055
4.0	.913	.0544	.650	.0382	.430	.0246	.209	.0117
4.8	1.292	.0862	.945	.0612	.638	.0399	.326	.0193
5.0	1.493	.0964	1.132	.0687	.806	.0451	.466	.0221
5.2	1.939	.1089	1.627	.0786	1.347	.0527	1.047	.0273
5.4	2.642	.1256	2.441	.0935	2.264	.0659	2.065	.0386
5.6	3.615	.1486	3.597	.1156	3.598	.0873	3.577	.0593
5.8	4.801	.1795	5.028	.1473	5.268	.1198	5.492	.0925
6.0	6.067	.2196	6.568	.1900	7.079	.1653	7.581	.1406
6.2	7.146	.2685	7.876	.2434	8.612	.2234	9.340	.2033
6.4	7.864	.3239	8.740	.3048	9.617	.2907	10.484	.2764
6.5	8.061	.3533	8.972	.3374	9.878	.3266	10.772	.3155
6.6	8.181	.3832	9.116	.3707	10.044	.3632	10.959	.3555
6.7	8.151	.4133	9.068	.4042	9.970	.4000	10.854	.3956
6.8	8.052	.4431	8.942	.4373	9.814	.4364	10.664	.4351
7.0	7.608	.5009	8.392	.5012	9.146	.5063	9.871	.5108
7.2	6.991	.5547	7.645	.5604	8.257	.5704	8.836	.5797
7.4	6.301	.6037	6.818	.6136	7.288	.6276	7.722	.6406
7.7	5.246	.6674	5.567	.6818	5.833	.6998	6.063	.7163
8.0	4.361	.7203	4.535	.7375	4.654	.7575	4.744	.7757
8.5	3.282	.7902	3.309	.8090	3.291	.8298	3.259	.8484
9.0	2.480	.8429	2.412	.8612	2.306	.8808	2.204	.8981
9.5	1.893	.8828	1.768	.8993	1.610	.9164	1.472	.9313
10.0	1.506	.9139	1.356	.9278	1.183	.9418	1.037	.9541
10.6	1.143	.9431	.980	.9535	.809	.9636	.662	.9726
11.2	.821	.9648	.680	.9717	.543	.9784	.420	.9844
12.0	.452	.9832	.371	.9869	.293	.9904	.224	.9936
13.0	.168	.9940	.130	.9956	.093	.9971	.059	.9985
14.0	.060	.9978	.044	.9984	.028	.9990	.013	.9995
17.0	0	1.0000	0	1.0000	0	1.0000	0	1.0000

Table 21.10.--(Continued)

T_c = 72 hours

Serial No. :	105		106		107		108	
Q_1/Q_{10} :	0.2		0.3		0.4		0.5	
Time	PSH	PSMC	PSH	PSMC	PSH	PSMC	PSH	PSMC
days	cfs/AQ_{10}	Q/Q_{10}	cfs/AQ_{10}	Q/Q_{10}	cfs/AQ_{10}	Q/Q_{10}	cfs/AQ_{10}	Q/Q_{10}
0	0	0	0	0	0	0	0	0
.6	.039	.0002	.029	.0002	.022	.0001	.017	.0001
1.3	.321	.0042	.241	.0031	.183	.0023	.143	.0018
2.0	.867	.0193	.660	.0146	.508	.0111	.395	.0086
3.0	1.552	.0650	1.230	.0500	.959	.0385	.762	.0302
4.0	2.001	.1311	1.668	.1037	1.349	.0810	1.101	.0644
4.8	2.345	.1953	2.076	.1586	1.777	.1266	1.497	.1021
5.0	2.452	.2130	2.226	.1745	1.950	.1403	1.676	.1137
5.2	2.587	.2317	2.438	.1917	2.226	.1556	2.001	.1272
5.4	2.753	.2514	2.725	.2107	2.620	.1735	2.492	.1437
5.6	2.946	.2725	3.078	.2322	3.123	.1946	3.136	.1643
5.8	3.161	.2951	3.487	.2564	3.721	.2198	3.919	.1903
6.0	3.383	.3193	3.923	.2838	4.372	.2497	4.785	.2224
6.2	3.591	.3451	4.338	.3143	5.000	.2843	5.629	.2608
6.4	3.755	.3723	4.658	.3476	5.477	.3230	6.257	.3047
6.6	3.863	.4006	4.866	.3829	5.782	.3646	6.655	.3524
6.7	3.895	.4149	4.923	.4010	5.862	.3861	6.755	.3771
6.8	3.911	.4294	4.948	.4192	5.894	.4078	6.790	.4021
6.9	3.912	.4438	4.943	.4375	5.879	.4295	6.762	.4271
7.0	3.899	.4583	4.908	.4557	5.819	.4511	6.672	.4518
7.2	3.838	.4869	4.772	.4915	5.601	.4933	6.364	.5000
7.5	3.689	.5288	4.460	.5428	5.121	.5528	5.712	.5669
8.0	3.378	.5942	3.844	.6195	4.210	.6388	4.508	.6610
8.5	3.088	.6540	3.303	.6853	3.454	.7091	3.550	.7347
9.0	2.840	.7088	2.866	.7422	2.874	.7673	2.836	.7934
9.5	2.628	.7593	2.507	.7918	2.418	.8160	2.284	.8404
10.0	2.450	.8062	2.220	.8353	2.064	.8572	1.863	.8783
10.6	2.222	.8583	1.925	.8813	1.719	.8990	1.491	.9153
11.2	1.836	.9037	1.551	.9200	1.337	.9330	1.131	.9444
12.0	1.142	.9478	.954	.9570	.804	.9644	.673	.9708
13.0	.523	.9774	.435	.9816	.365	.9851	.305	.9880
14.0	.229	.9906	.185	.9925	.150	.9941	.119	.9954
15.0	.098	.9963	.078	.9971	.062	.9977	.048	.9983
17.0	0	1.0000	0	1.0000	0	1.0000	0	1.0000

Table 21.10.--(Continued)

T_c = 72 hours

Serial No. :	109		110		111		112	
Q_1/Q_{10} :	0.6		0.7		0.8		0.9	
Time	PSH	PSMC	PSH	PSMC	PSH	PSMC	PSH	PSMC
days	cfs/AQ_{10}	Q/Q_{10}	cfs/AQ_{10}	Q/Q_{10}	cfs/AQ_{10}	Q/Q_{10}	cfs/AQ_{10}	Q/Q_{10}
0	0	0	0	0	0	0	0	0
.6	.013	.0001	.009	.0000	.005	.0000	.002	.0000
1.3	.108	.0014	.075	.0010	.044	.0006	.021	.0003
2.0	.300	.0065	.210	.0045	.128	.0027	.062	.0013
3.0	.581	.0229	.408	.0160	.266	.0100	.125	.0048
4.0	.856	.0492	.608	.0345	.402	.0222	.194	.0105
4.8	1.202	.0789	.876	.0559	.590	.0364	.299	.0176
5.0	1.378	.0884	1.037	.0629	.733	.0412	.419	.0201
5.2	1.744	.0997	1.437	.0717	1.164	.0478	.877	.0245
5.4	2.324	.1146	2.103	.0847	1.910	.0590	1.701	.0338
5.6	3.106	.1345	3.019	.1034	2.956	.0768	2.874	.0505
5.8	4.072	.1609	4.171	.1298	4.288	.1033	4.388	.0771
6.0	5.156	.1948	5.478	.1653	5.814	.1405	6.136	.1157
6.2	6.221	.2368	6.771	.2104	7.334	.1889	7.888	.1673
6.4	7.001	.2856	7.705	.2638	8.413	.2469	9.111	.2299
6.6	7.491	.3391	8.290	.3228	9.085	.3114	9.867	.2999
6.7	7.610	.3669	8.427	.3536	9.237	.3451	10.032	.3365
6.8	7.647	.3950	8.467	.3847	9.275	.3792	10.065	.3734
6.9	7.604	.4231	8.411	.4157	9.200	.4132	9.969	.4102
7.0	7.483	.4508	8.258	.4464	9.010	.4467	9.739	.4465
7.2	7.083	.5046	7.767	.5055	8.420	.5109	9.044	.5157
7.5	6.257	.5784	6.775	.5860	7.251	.5976	7.693	.6082
8.0	4.767	.6797	5.015	.6942	5.213	.7118	5.380	.7278
8.5	3.622	.7563	3.698	.7735	3.728	.7930	3.742	.8104
9.0	2.792	.8151	2.765	.8327	2.699	.8518	2.632	.8686
9.5	2.166	.8606	2.075	.8770	1.950	.8943	1.839	.9095
10.0	1.700	.8958	1.568	.9101	1.411	.9247	1.275	.9376
10.6	1.311	.9290	1.161	.9401	1.001	.9511	.863	.9609
11.2	.962	.9540	.822	.9619	.683	.9695	.559	.9764
12.0	.561	.9762	.470	.9806	.381	.9849	.301	.9887
13.0	.253	.9906	.212	.9927	.172	.9947	.136	.9965
14.0	.092	.9966	.069	.9975	.047	.9984	.027	.9992
15.0	.036	.9987	.026	.9991	.016	.9994	.008	.9997
17.0	0	1.0000	0	1.0000	0	1.0000	0	1.0000

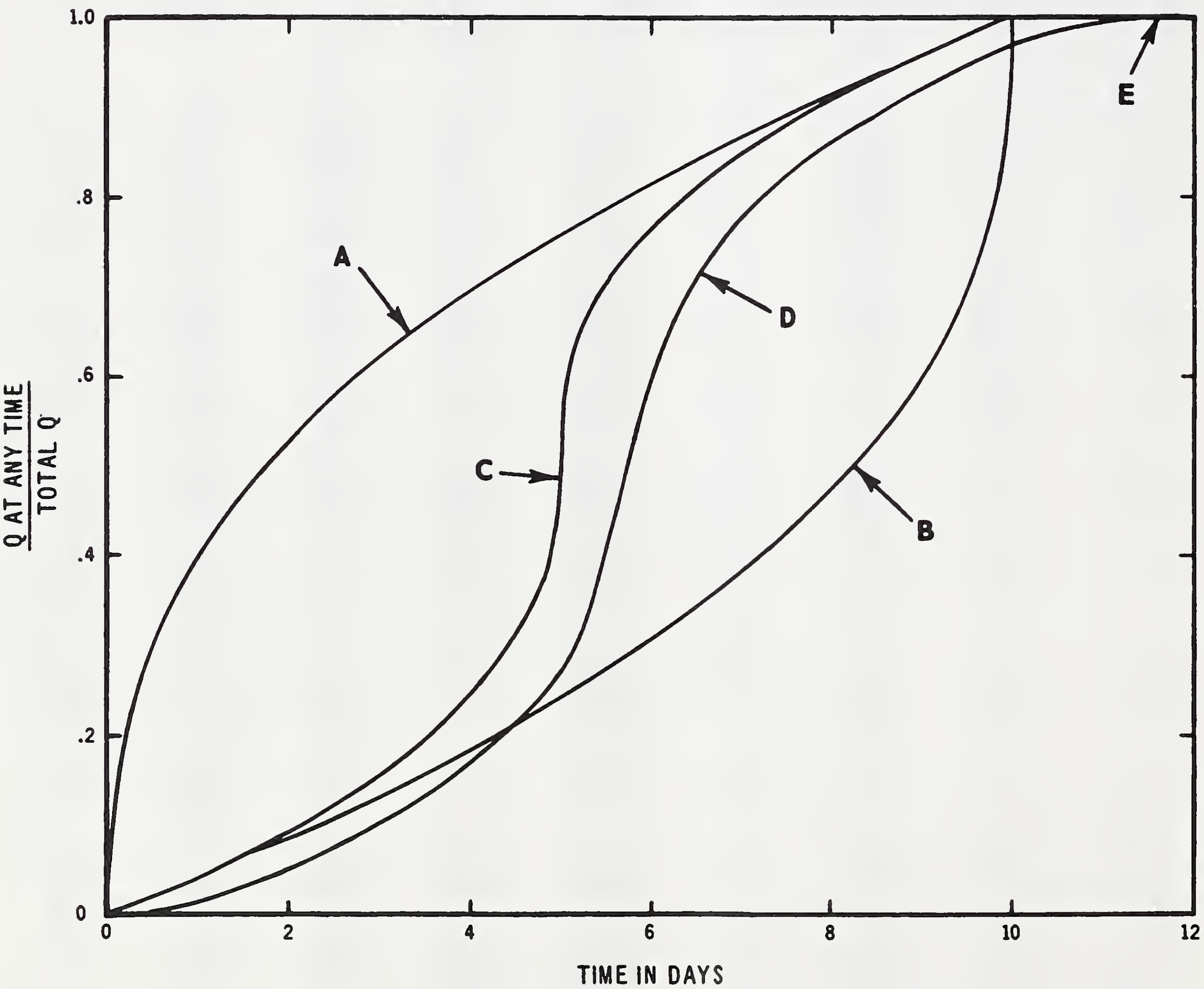
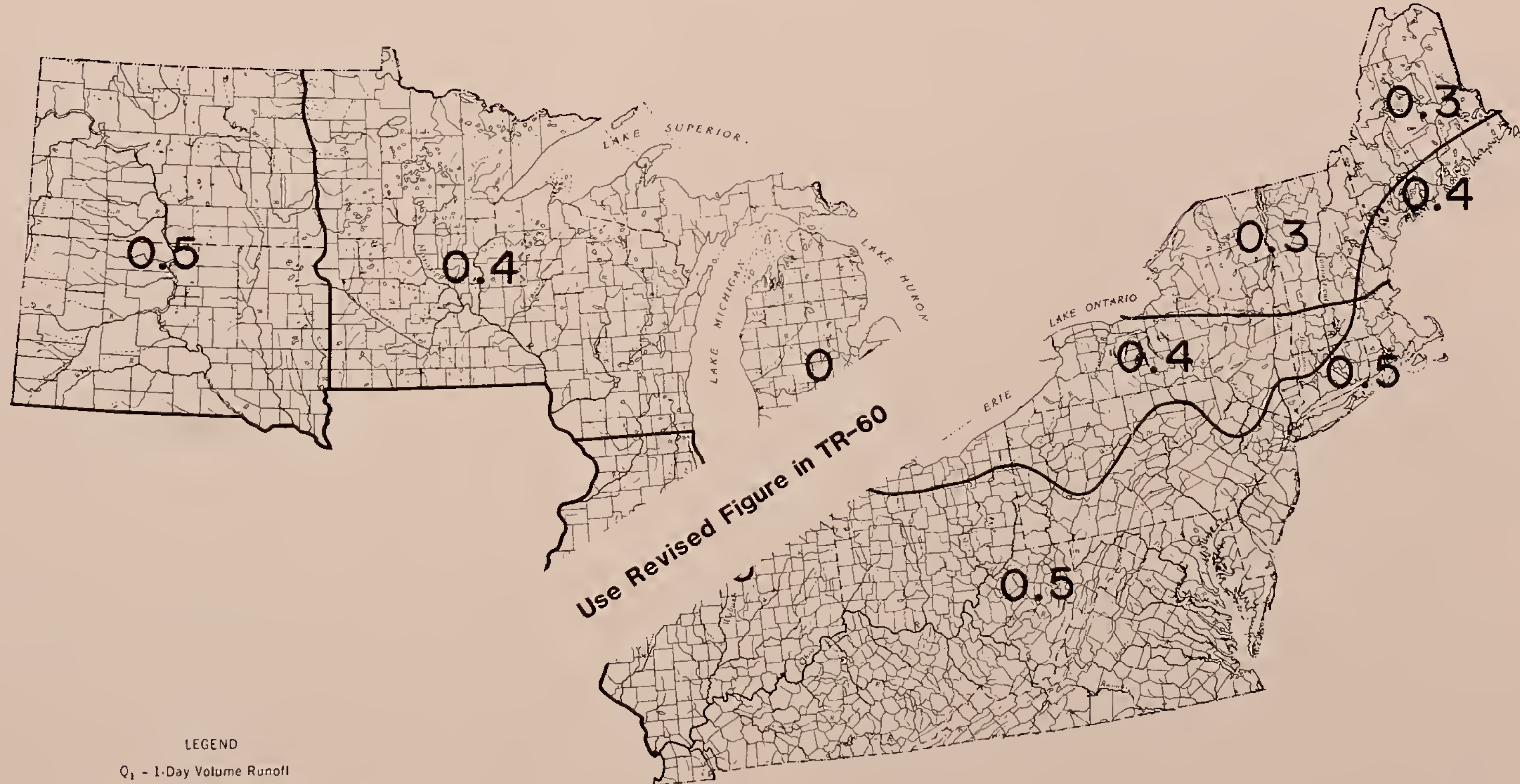


FIGURE 21.1 - Mass curves of runoff in various arrangements.

RATIOS FOR 50 AND 25-YEAR 10-DAY RUNOFF VOLUMES

To obtain:	Multiply map values by: Area ①	Area ②	Area ③
50-YEAR 10-DAY RUNOFF	0.85	0.90	0.92
25-YEAR 10-DAY RUNOFF	0.70	0.80	0.85

Exhibit 21-1 100-YEAR 10-DAY RUNOFF (Inches) for developing the Principal Spillway Hydrograph



LEGEND

Q_1 - 1-Day Volume Runoff
Q_{10} - 10-Day Volume Runoff

Exhibit 21-2 RATIOS OF VOLUMES OF RUNOFF (Q_1/Q_{10}) for developing the Principal Spillway Hydrograph

(210-VI-NEH-4, Amend. 6, March 1985)

Note: For Wisconsin and Michigan use csm values shown

Exhibit 21-3 QUICK-RETURN FLOW (csm) for developing the Principal Spillway Hydrograph

Exhibit 21-4 100-YEAR 10-DAY RUNOFF (Inches) for developing the Principal Spillway Hydrograph

JANUARY 1971

LEGEND

Primarily snowmelt produced floods

Primarily rain produced floods

Deep snowpack areas

Q_1 - 1-Day Volume Runoff

Q_{10} - 10-Day Volume Runoff

Exhibit 21-5 RATIOS OF VOLUMES OF RUNOFF (Q_1/Q_{10}) for developing the Principal Spillway Hydrograph

Emergency Spillways [1]

Flows larger than those completely controllable by the principal spillway and retarding storage are safely conveyed past an earth dam by an emergency spillway. The emergency spillway is designed by use of an Emergency Spillway Hydrograph (ESH) and its minimum freeboard determined by use of a Freeboard Hydrograph (FH). Both kinds of hydrographs are constructed by the same procedure. There is a small difference in that procedure depending on whether a watershed's time of concentration is or is not over six hours.

This part of the chapter presents a manual method of developing ESH and FH. The method requires the use of the dimensionless hydrographs given in table 21.17. Methods of routing the ESH or FH through structures are given in chapter 17.

Alternatives to developing and routing the hydrographs manually are (i) use of the SCS electronic computer program, in which basic data are input and the ESH or FH, the routed hydrograph, and reservoir elevations are output; and (ii) the Upper Darby or UD method, in which no hydrograph is needed but which uses the hydrograph characteristics of ESH or FH in an indirect routing procedure with results in terms of spillway elevation and capacity.

The hydrologic criteria given below apply to the manual method and its alternatives. The examples that follow apply only to the manual method.

Hydrologic Criteria

SOURCE OF DESIGN STORM RAINFALL AMOUNT. The basic 6-hour design storm rainfall amount used in development of ESH and FH is taken from one of the following maps:

1/ Background information on the material in this part of the chapter is given in "Central Technical Unit Method of Hydrograph Development," by M. H. Kleen and R. G. Andrews, Transactions, American Society of Agricultural Engineers, vol. 5, no. 2, p. 180-185, 1962; and in "Hydrology of Spillway Design: Small Structures - Limited Data," by Harold O. Ogrosky, paper no. 3914, Proceedings, American Society of Civil Engineers, Journal of the Hydraulics Division, May 1964.

ES-1020, 5 sheets. 48 contiguous States. Supplementary sheets for California and Washington-Oregon are also given.

ES-1021, 5 sheets. Hawaii.

ES-1022, 5 sheets. Alaska.

ES-1023, 5 sheets. Puerto Rico.

ES-1024, 5 sheets. Virgin Islands.

The rainfall amounts on these maps are minimums allowed by SCS criteria for various classes of structures.

DURATION ADJUSTMENT OF RAINFALL AMOUNT. If the time of concentration of the drainage area above a structure is more than six hours, the duration of the design storm is made equal to that time and the rainfall amount is increased using a factor from figure 21.2, part (c).

AREAL ADJUSTMENT OF RAINFALL AMOUNT. If the drainage area above a structure is 10 square miles or less, the areal rainfall is the same as the rainfall taken from the maps of ES-1020 through 1024. If the area is over 10 square miles but not over 100 square miles, the areal rainfall is obtained by use of a factor from figure 21.2, part (a). If the area is over 100 square miles, the adjustment factor for the area is requested from the Engineering Division, Washington, D. C. When a request is submitted, the following information about the area should also be submitted: (1) location, preferably the latitude and longitude of the watershed outlet; (2) size in square miles; (3) length in miles, following the main valley; (4) time of concentration in hours; (5) runoff curve number; (6) proposed value of the adjustment or adjustment factor. If a factor is also needed for a subwatershed of that watershed, then similar information about the subwatershed should also be submitted.

RUNOFF DETERMINATION. Runoff is determined using the methods of chapter 10. The runoff curve number (CN) for the drainage area above a structure is determined by any of the methods in chapter 10. This CN must be for antecedent moisture condition II or greater and it applies throughout the design storm regardless of the storm duration.

DIMENSIONLESS HYDROGRAPHS. The ESH and FH are made using the dimensionless hydrographs given in table 21.17. If a hydrograph is to be developed in an electronic computer program, then the storm distribution given in figure 21.2.b (ES-1003-b) must be used to get an equivalent ESH or FH.

Construction of Emergency Spillway and Freeboard Hydrographs

Two examples of hydrograph construction are given. The first illustrates the procedure when the watershed time of concentration is not over six hours,

the second when it is. There is no difference in procedure for ESH and FH. Equations used in the examples are listed in table 21.11.

Example 21.5.--Construct an ESH for a class (b) structure with a drainage area of 1.86 square miles, time of concentration of 1.25 hours, CN of 82, and location at latitude____, longitude____.

1. Determine the 6-hour design storm rainfall amount, P. For this structure class the ESH rainfall amount is taken from ES-1020, sheet 2 of 5. For the given location the map shows that P = 9.4 inches.

2. Determine the areal rainfall amount. The areal rainfall is the same as in step 1 because the drainage area is not over 10 square miles. Step 2 of example 21.6 shows the process.

3. Make the duration adjustment of rainfall amount. No adjustment is made because the time of concentration is not over six hours. Step 3 of example 21.6 shows the process.

4. Determine the runoff amount, Q. Enter figure 10.1 with P = 9.4 inches and CN = 82 and find Q = 7.21 inches.

5. Determine the hydrograph family. Enter figure 21.3 (ES-1011) with CN = 82 and at P = 9.4 read hydrograph family 2.

6. Determine the duration of excess rainfall, T_o. Enter figure 21.4 (ES-1012) with P = 9.4 inches and at CN = 82 read by interpolation that T_o = 5.37 hours.

7. Compute the initial value of T_p. By equation 21.4 this is 0.7(1.25) = 0.88 hours.

8. Compute the T_o/T_p ratio. This is 5.37/0.88 = 6.10.

9. Select a revised T_o/T_p ratio from table 21.16. This table shows the hydrograph families and ratios for which dimensionless hydrographs are given in table 21.17. Enter table 21.16 with the ratio from step 8 and select the tabulated ratio nearest it. For this example the selected ratio, (T_o/T_p)rev., is 6.

10. Compute Rev. T_p. This is a revised T_p used because of the change in ratio. By equation 21.5, Rev. T_p = 5.37/6 = 0.895 hours.

11. Compute q_p. By equation 21.6 this is 484(1.86)/0.895 = 1006 cfs.

12. Compute Q_{qp}. Using the Q from step 4 and the q_p from step 11 gives $Q(q_p)$ = 7.21(1006) = 7253.26 cfs. Round to 7250 cfs.

13. Compute the times for which hydrograph rates will be computed. In equation 21.7 use Rev. T_p from step 10 and the entries in the t/T_p column of the selected hydrograph in table 21.17. The computed times are shown in column 2 of table 21.12.

Table 21.11--Equations used in construction of ESH and FH

Equation	No.
$T_p = 0.7\ T_c$	21.4
$\text{Rev. } T_p = \dfrac{T_o}{(T_o/T_p)_{rev.}}$	21.5
$q_p = \dfrac{484\ A}{\text{Rev. } T_p}$	21.6
$t = (t/T_p)\ (\text{Rev. } T_p)$	21.7
$q = (q_c/q_p)\ Q_{qp}$	21.8

where
- A = drainage area in square miles
- q = hydrograph rate in cfs
- q_c = hydrograph rate in cfs when $Q = 1$ inch
- q_p = hydrograph peak rate in cfs when $Q = 1$ inch
- Q = design storm runoff in inches
- Rev. T_p = revised time to peak in hours
- t = time in hours at which hydrograph rate is computed
- T_c = time of concentration in hours
- T_o = duration of excess rainfall in hours
- $(T_o/T_p)_{rev.}$ = revised ratio from table 21.16
- T_p = time to peak in hours for CTU design hydrographs

14. Compute the hydrograph rates. Use equation 21.8 and the q_c/q_p column of the selected hydrograph in table 21.17. The computed rates are shown in column 3 of table 21.12.

The hydrograph is completed with step 14. How the hydrograph is further retabulated or plotted for routing through the spillway depends on the routing method to be used. See chapter 17 for routing details.

The mass curve for the hydrograph can be obtained using the Q_t/Q column of the selected hydrograph in table 21.17. Ratios in that column are multiplied by the Q of step 4 to give accumulated runoff in inches at the time computed in step 13. For accumulated runoff in acre-feet or another unit, convert Q to the desired unit before making the series of multiplications.

In the following example the storm duration is increased because the time of concentration is over six hours. Increasing the duration also requires increasing the rainfall amount but if the drainage area is over 10 square miles the increase is partly offset by the decrease in areal rainfall.

Example 21.6.--Construct a FH for a class (c) structure with a drainage area of 23.0 square miles, time of concentration of 10.8 hours, CN of 77, and location at latitude____, longitude____.

1. Determine the 6-hour design storm rainfall amount, P. For this structure class the FH rainfall amount is taken from ES-1020, sheet 5 of 5. For the given location the map shows that P = 25.5 inches.

2. Determine the areal rainfall amount. Use the appropriate curve on figure 21.2.a (ES-1003-a). For this location the "Humid and subhumid climate" curve applies and the adjustment factor for the drainage area of 23.0 square miles is 0.93. The adjusted rainfall is 0.93(25.5) = 23.72 inches.

3. Make the duration adjustment of rainfall amount. The duration is made equal to the time of concentration, in this case, 10.8 hours. Enter figure 21.2.c (ES-1003-c) with the duration of 10.8 hours and find an adjustment factor of 1.18. The adjusted rainfall is 1.18(23.72) = 27.99 inches. It is rounded to 28.0 inches for the remainder of this example.

4. Determine the runoff amount, Q. Enter figure 10.1 with the rainfall from step 3 (P = 28.0 inches) and at CN = 77 find Q = 24.7 inches.

5. Determine the hydrograph family. Enter figure 21.3 (ES-1011) with CN = 77 and at P = 28.0 inches read hydrograph family 1.

6. Determine the duration of excess rainfall, T_o. Enter table 21.14 with CN = 77 and find that P*, the rainfall prior to the excess rainfall, is 0.60 inches. Enter table 21.15 with the ratio P*/P = 0.60/28.0 = 0.0214 and by interpolation read a time ratio of 0.950. Then T_o = (time ratio) x (storm duration) = 0.950(10.8) = 10.26 hours.

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HYDROGRAPH COMPUTATION

DATE ~~~
COMPUTED BY ~~~
CHECKED BY ~~~

WATERSHED OR PROJECT (EXAMPLE 21.5)

STATE ~~~

STRUCTURE SITE OR SUBAREA ~~~

DR. AREA 1.86 SQ. MI. STRUCTURE CLASS b

T_c 1.25 HR. STORM DURATION 6 HR.

POINT RAINFALL 9.4 IN.

ADJUSTED RAINFALL:

AREAL : FACTOR 1.0 IN. 9.4

DURATION : FACTOR 1.0 IN. 9.4

RUNOFF CURVE NO. 82

Q 7.21 IN.

HYDROGRAPH FAMILY NO. 2

COMPUTED T_p 0.88 HR.

T_o 5.37 HR.

(T_o / T_p):
COMPUTED 6.10 ; USED 6

REVISED T_p 0.895

$q_p = \frac{484A}{\text{REV. } T_p} =$ 1006 CFS.

$(Q)(q_p) =$ 7250 CFS.

t(COLUMN) = (t/T_p) REV. T_p q(COLUMN) = $(q_c/q_p)(Q)(q_p)$

Q(COLUMN) = $(Q_t/Q)Q$

	$t=(t/T_p)$Rev. T_p	$q=(q_c/q_p)(Q)(q_p)$	$Q_t=(Q_t/Q)Q$
	t HOURS	q CFS	Q INCHES
1	0	0	0
2	.30	7	
3	.61	36	
4	.91	109	
5	1.22	268	
6	1.52	710	
7	1.82	1769	
8	2.13	2951	
9	2.43	3364	
10	2.74	3110	
11	3.04	2661	
12	3.35	2240	
13	3.65	1892	
14	3.96	1624	
15	4.26	1399	
16	4.56	1225	
17	4.87	1102	
18	5.17	1008	
19	5.48	935	
20	5.78	819	
21	6.09	616	
22	6.39	399	
23	6.69	254	
24	7.00	145	
25	7.30	87	
26	7.61	58	
27	7.91	36	
28	8.22	29	
29	8.52	22	
30	8.82	14	
31	9.13	7	
32	9.43	0	
33			
34			

Table 21.12 Hydrograph computation

7. Compute the initial value of T_p. By equation 21.4 this is 0.7(10.8) = 7.56 hours.

8. Compute the T_o/T_p ratio. This is 10.26/7.56 = 1.357.

9. Select a revised T_o/T_p ratio from table 21.16. Enter table 21.16 with the ratio from step 8 and select the tabulated ratio nearest it. For this example the selected ratio, (T_o/T_p)rev., is 1.5.

10. Compute Rev. T_p. This is a revised T_p used because of the change in ratio. By equation 21.5, Rev. T_p = 10.26/1.5 = 6.84 hours.

11. Compute q_p. By equation 21.6 this is 484(23.0)/6.84 = 1627.5 cfs. Round to 1628 cfs.

12. Compute Qq_p. Using the Q from step 4 and the q_p from step 11 gives $Q(q_p)$ = 24.7(1628) = 40,211.6 cfs. Round to 40,212 cfs.

13. Compute the times for which hydrograph rates will be computed. Use equation 21.7 with the Rev. T_p from step 10 and the entries in the t/T_p column of the selected hydrograph in table 21.17. The computed rates are shown in column 2 of table 21.13.

14. Compute the hydrograph rates. Use equation 21.8 with Qq_p of step 12 and the q_c/q_p column of the selected hydrograph in table 21.17. The computed rates are shown in column 3 of table 21.13.

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HYDROGRAPH COMPUTATION

DATE ______
COMPUTED BY ______
CHECKED BY ______

WATERSHED OR PROJECT (EXAMPLE 21.6)

STATE ______

STRUCTURE SITE OR SUBAREA ______

DR. AREA 23.0 SQ. MI. STRUCTURE CLASS C

T_c 10.8 HR. STORM DURATION 10.8 HR.

POINT RAINFALL 25.5 IN.

ADJUSTED RAINFALL:

AREAL: FACTOR .93 IN. 23.72

DURATION: FACTOR 1.18 IN. 27.99

RUNOFF CURVE NO. 77

Q 24.7 IN.

HYDROGRAPH FAMILY NO. 1

COMPUTED T_p 7.56 HR.

T_o 10.26 HR.

(T_o / T_p):

COMPUTED 1.357 ; USED 1.5

REVISED T_p 6.84

$q_p = \frac{484A}{REV.\ T_p} =$ 1628 CFS.

$(Q)(q_p) =$ 40,212 CFS.

t(COLUMN) = (t / T_p) REV. T_p

q(COLUMN) = $(q_c / q_p)(Q)(q_p)$

Q(COLUMN) = $(Q_t / Q)Q$

	$t=(t/T_p)$Rev. T_p	$q=(q_c/q_p)(Q)(q_p)$	$Q_t=(Q_t/Q)Q$
	t HOURS	q CFS	Q INCHES
1	0	0	0
2	2.19	482	
3	4.38	4745	
4	6.57	15160	
5	8.76	28591	
6	10.94	32773	
7	13.13	28912	
8	15.32	21152	
9	17.51	14155	
10	19.70	9048	
11	21.89	5750	
12	24.08	3619	
13	26.26	2292	
14	28.45	1488	
15	30.64	965	
16	32.83	603	
17	35.02	322	
18	37.21	161	
19	39.40	80	
20	41.59	40	
21	43.78	0	
22			
23			
24			
25			
26			
27			
28			
29			
30			
31			
32			
33			
34			

Table 21.13 Hydrograph computation.

Table 21.14.--Rainfall prior to excess rainfall.

CN	P*	CN	P*	CN	P*	CN	P*	CN	P*
	(inches)		(inches)		(inches)		(inches)		(inches)
100	0	86	0.33	72	0.78	58	1.45	44	2.54
99	.02	85	.35	71	.82	57	1.51	43	2.64
98	.04	84	.38	70	.86	56	1.57	42	2.76
97	.06	83	.41	69	.90	55	1.64	41	2.88
96	.08	82	.44	68	.94	53	1.70	40	3.00
95	.11	81	.47	67	.98	53	1.77	39	3.12
94	.13	80	.50	66	1.03	52	1.85	38	3.26
93	.15	79	.53	65	1.08	51	1.92	37	3.40
92	.17	78	.56	64	1.12	50	2.00	36	3.56
91	.20	77	.60	63	1.17	49	2.08	35	3.72
90	.22	76	.63	62	1.23	48	2.16	34	3.88
89	.25	75	.67	61	1.28	47	2.26	33	4.06
88	.27	74	.70	60	1.33	46	2.34	32	4.24
87	.30	73	.74	59	1.39	45	2.44	31	4.44

Table 21.15.--Rainfall and time ratios for determining T_o when the storm duration is greater than 6 hours.

Rain-fall ratio	Time ratio	Rain-fall ratio	Time ratio	Rain-fall ratio	Time ratio	Rain-fall ratio	Time ratio
0	1.000	0.070	0.852	0.140	0.746	0.210	0.684
.002	.995	.072	.848	.142	.744	.212	.682
.004	.990	.074	.844	.144	.742	.214	.680
.006	.985	.076	.841	.146	.740	.216	.679
.008	.981	.078	.837	.148	.739	.218	.677
.010	.976	.080	.833	.150	.737	.220	.675
.012	.971	.082	.830	.152	.735	.222	.673
.014	.967	.084	.827	.154	.733	.224	.672
.016	.962	.086	.824	.156	.732	.226	.670
.018	.957	.088	.821	.158	.730	.228	.668
.020	.952	.090	.818	.160	.728	.230	.667
.022	.948	.092	.815	.162	.726	.232	.666
.024	.943	.094	.812	.164	.724	.234	.666
.026	.938	.096	.809	.166	.723	.236	.665
.028	.933	.098	.806	.168	.721	.238	.665
.030	.929	.100	.803	.170	.719	.240	.664
.032	.924	.102	.800	.172	.717		
.034	.919	.104	.797	.174	.716	(Change in tabulation increment.)	
.036	.915	.106	.794	.176	.714		
.038	.911	.108	.791	.178	.712		
.040	.908	.110	.788	.180	.710	.250	.662
.042	.904	.112	.785	.182	.709	.300	.651
.044	.900	.114	.782	.184	.707	.350	.640
.046	.896	.116	.779	.186	.705	.400	.628
.048	.893	.118	.776	.188	.703	.450	.617
.050	.889	.120	.773	.190	.702	.500	.606
.052	.885	.122	.770	.192	.700	.550	.595
.054	.882	.124	.767	.194	.698	.600	.583
.056	.878	.126	.764	.196	.696	.650	.542
.058	.874	.128	.761	.198	.695	.700	.500
.060	.870	.130	.758	.200	.693	.750	.447
.062	.867	.132	.755	.202	.691	.800	.386
.064	.863	.134	.751	.204	.689	.850	.310
.066	.859	.136	.749	.206	.687	.900	.220
.068	.856	.138	.747	.208	.686	.950	.116

Table 21.16.--Hydrograph families and T_o/T_p ratios for which dimensionless hydrograph ratios are given in table 21.17

Hydrograph Family	T_o/T_p 1	1.5	2	3	4	6	10	16	25	36	50	75
1	*	*	*	*	*	*	*	*	*	*	*	*
2	*	*	*	*	*	*	*	*	*	*	*	*
3	*	*	*	*	*	*	*	*	*	*	*	*
4	*	*	*	*	*	*	*	*	*	*	*	
5	*	*	*	*	*	*	*	*	*	*	*	

Asterisks signify that dimensionless hydrograph tabulations are given in table 21.17.

Table 21.17 --Time, discharge, and accumulated runoff ratios for dimensionless hydrographs

Hydrograph Family 1

	$T_o/T_p = 1$			$T_o/T_p = 1.5$			$T_o/T_p = 2$		
Line No.	t/T_p	q_c/q_p	Q_t/Q	t/T_p	q_c/q_p	Q_t/Q	t/T_p	q_c/q_p	Q_t/Q
1	0	0	0	0	0	0	0	0	0
2	.28	.029	.003	.32	.012	.001	.29	.007	.001
3	.56	.150	.021	.64	.118	.017	.58	.035	.005
4	.84	.472	.086	.96	.377	.075	.87	.164	.027
5	1.12	.798	.216	1.28	.711	.204	1.16	.432	.090
6	1.40	.901	.392	1.60	.815	.384	1.45	.669	.208
7	1.68	.776	.564	1.92	.719	.565	1.74	.740	.359
8	1.96	.568	.703	2.24	.526	.712	2.03	.680	.511
9	2.24	.389	.801	2.56	.352	.815	2.32	.561	.644
10	2.52	.258	.868	2.88	.225	.884	2.61	.441	.751
11	2.80	.173	.913	3.20	.143	.927	2.90	.319	.833
12	3.08	.115	.942	3.52	.090	.954	3.19	.212	.890
13	3.36	.078	.962	3.84	.057	.972	3.48	.140	.927
14	3.64	.052	.976	4.16	.037	.983	3.77	.094	.952
15	3.92	.036	.985	4.48	.024	.990	4.06	.063	.969
16	4.20	.024	.991	4.80	.015	.995	4.35	.042	.981
17	4.48	.016	.995	5.12	.008	.997	4.64	.028	.988
18	4.76	.009	.997	5.44	.004	.999	4.93	.017	.993
19	5.04	.005	.999	5.76	.002	1.000	5.22	.011	.996
20	5.32	.002	1.000	6.08	.001	1.000	5.51	.007	.998
21	5.60	.001	1.000	6.40	0	1.000	5.80	.004	.999
22	5.88	0	1.000				6.09	.002	1.000
23							6.38	.001	1.000
24							6.67	0	1.000

Table 21.17 (Continued) Hydrograph Family 1

	$T_o/T_p = 3$			$T_o/T_p = 4$			$T_o/T_p = 6$		
Line No.	t/T_p	q_c/q_p	Q_t/Q	t/T_p	q_c/q_p	Q_t/Q	t/T_p	q_c/q_p	Q_t/Q
1	0	0	0	0	0	0	0	0	0
2	.35	.005	.001	.35	.003	.000	.44	.003	.001
3	.70	.027	.005	.70	.015	.003	.98	.018	.003
4	1.05	.101	.021	1.05	.049	.011	1.32	.041	.012
5	1.40	.302	.074	1.40	.122	.033	1.76	.084	.032
6	1.75	.563	.185	1.75	.298	.087	2.20	.176	.074
7	2.10	.650	.342	2.10	.528	.194	2.64	.386	.165
8	2.45	.576	.501	2.45	.585	.337	3.08	.497	.309
9	2.80	.460	.635	2.80	.518	.479	3.52	.430	.459
10	3.15	.374	.743	3.15	.413	.599	3.96	.335	.583
11	3.50	.290	.829	3.50	.334	.695	4.40	.258	.679
12	3.85	.201	.892	3.85	.273	.774	4.84	.202	.754
13	4.20	.127	.935	4.20	.231	.839	5.28	.164	.813
14	4.55	.078	.961	4.55	.185	.892	5.72	.139	.862
15	4.90	.047	.977	4.90	.128	.933	6.16	.124	.905
16	5.25	.028		5.25	.080	.959	6.60	.100	.941
17	5.60	.016	.993	5.60	.047	.976	7.04	.060	.967
18	5.95	.009	.996	5.95	.028	.985	7.48	.033	.982
19	6.30	.005	.998	6.30	.017	.991	7.92	.018	.991
20	6.65	.003	.999	6.65	.010	.995	8.36	.009	.995
21	7.00	.002	.999	7.00	.006	.997	8.80	.005	.997
22	7.35	.001	1.000	7.35	.004	.998	9.24	.003	.999
23	7.70	0	1.000	7.70	.003	.999	9.68	.002	.999
24				8.05	.002	1.000	10.12	.001	1.000
25				8.40	.001	1.000	10.56	0	1.000
26				8.75	0	1.000			

Table 21.17 (Continued) Hydrograph Family 1

Line No.	$T_o/T_p = 10$ t/T_p	q_c/q_p	Qt/Q	$T_o/T_p = 16$ t/T_p	q_c/q_p	Qt/Q	$T_o/T_p = 25$ t/T_p	q_c/q_p	Qt/Q
1	0	0	0	0	0	0	0	0	0
2	.56	.002	.000	.66	.001	.000	1.22	.002	.001
3	1.12	.013	.004	1.32	.006	.002	2.44	.009	.006
4	1.68	.027	.012	1.98	.015	.007	3.66	.018	.018
5	2.24	.047	.027	2.64	.027	.017	4.88	.027	.038
6	2.80	.071	.052	3.30	.037	.033	6.10	.036	.067
7	3.36	.115	.090	3.96	.047	.053	7.32	.046	.103
8	3.92	.278	.172	4.62	.062	.080	8.54	.116	.176
9	4.48	.394	.312	5.28	.092	.117	9.76	.232	.333
10	5.04	.322	.461	5.94	.223	.194	10.98	.146	.503
11	5.60	.235	.577	6.60	.309	.323	12.20	.088	.608
12	6.16	.174	.662	7.26	.243	.457	13.42	.062	.675
13	6.72	.136	.726	7.92	.171	.557	14.64	.051	.726
14	7.28	.110	.777	8.58	.124	.629	15.86	.045	.769
15	7.84	.092	.819	9.24	.097	.683	17.08	.039	.807
16	8.40	.079	.855	9.90	.081	.726	18.30	.035	.840
17	8.96	.073	.886	10.56	.070	.763	19.52	.031	.870
18	9.52	.068	.916	11.22	.061	.794	20.74	.027	.896
19	10.08	.065	.943	11.88	.055	.823	21.96	.025	.920
20	10.64	.053	.968	12.54	.050	.848	23.18	.025	.942
21	11.20	.027	.984	13.20	.047	.872	24.40	.025	.965
22	11.76	.012	.993	13.86	.045	.894	25.62	.020	.985
23	12.32	.006	.996	14.52	.044	.916	26.84	.005	.996
24	12.88	.003	.998	15.18	.043	.937	28.06	.002	.999
25	13.44	.002	.999	15.84	.040	.957	29.28	0	1.000
26	14.00	.001	1.000	16.50	.034	.975			
27	14.56	0	1.000	17.16	.020	.988			
28				17.82	.008	.995			
29				18.48	.004	.998			
30				19.14	.002	.999			
31				19.80	.001	1.000			
32				20.46	0	1.000			

Table 21.17 (Continued) Hydrograph Family 1

	$T_o/T_p = 36$			$T_o/T_p = 50$			$T_o/T_p = 75$		
Line No.	t/T_p	q_c/q_p	Q_t/Q	t/T_p	q_c/q_p	Q_t/Q	t/T_p	q_c/q_p	Q_t/Q
1	0	0	0	0	0	0	0	0	0
2	1.70	.002	.001	2.00	.0019	.001	3.00	.0017	.002
3	3.40	.008	.008	4.00	.0052	.007	6.00	.0039	.008
4	5.10	.014	.021	6.00	.0085	.017	9.00	.0054	.018
5	6.80	.020	.043	8.00	.0118	.031	12.00	.0084	.033
6	8.50	.026	.072	10.00	.0151	.051	15.00	.0106	.053
7	10.20	.033	.109	12.00	.0192	.076	18.00	.0137	.079
8	11.90	.077	.178	14.00	.0259	.109	21.00	.0197	.115
9	13.60	.177	.338	16.00	.0578	.170	24.00	.0516	.192
10	15.30	.101	.513	18.00	.1330	.310	27.00	.0900	.344
11	17.00	.058	.613	20.00	.0941	.475	30.00	.0593	.504
12	18.70	.044	.678	22.00	.0506	.581	33.00	.0321	.602
13	20.40	.036	.728	24.00	.0357	.644	36.00	.0226	.661
14	22.10	.030	.770	26.00	.0297	.692	39.00	.0188	.705
15	23.80	.027	.805	28.00	.0254	.732	42.00	.0161	.742
16	25.50	.024	.838	30.00	.0219	.766	45.00	.0142	.775
17	27.20	.022	.867	32.00	.0192	.797	48.00	.0125	.804
18	28.90	.020	.893	34.00	.0172	.823	51.00	.0112	.829
19	30.60	.018	.917	36.00	.0159	.847	54.00	.0105	.852
20	32.30	.017	.939	38.00	.0150	.870	57.00	.0100	.874
21	34.00	.017	.960	40.00	.0145	.891	60.00	.0097	.896
22	35.70	.017	.982	42.00	.0140	.912	63.00	.0094	.916
23	37.40	.004	.995	44.00	.0136	.932	66.00	.0090	.936
24	39.10	.002	.999	46.00	.0131	.952	69.00	.0087	.955
25	40.80	0	1.000	48.00	.0125	.971	72.00	.0084	.973
26				50.00	.0123	.989	75.00	.0081	.991
27				52.00	.0016	.999	78.00	.0002	1.000
28				54.00	0	1.000	81.00	0	1.000

Table 21.17 (Continued)

Hydrograph Family 2

Line No.	$T_o/T_p = 1$ t/T_p	q_c/q_p	Q_t/Q	$T_o/T_p = 1.5$ t/T_p	q_c/q_p	Q_t/Q	$T_o/T_p = 2$ t/T_p	q_c/q_p	Q_t/Q
1	0	0	0	0	0	0	0	0	0
2	.28	.026	.003	.22	.003	.000	.28	.004	.000
3	.56	.170	.023	.44	.041	.004	.56	.040	.005
4	.84	.480	.091	.66	.161	.020	.84	.170	.027
5	1.12	.802	.224	.88	.362	.063	1.12	.428	.089
6	1.40	.885	.399	1.10	.604	.142	1.40	.645	.200
7	1.68	.770	.571	1.32	.740	.251	1.68	.715	.340
8	1.96	.550	.708	1.54	.790	.375	1.96	.677	.484
9	2.24	.380	.804	1.76	.746	.501	2.24	.574	.614
10	2.52	.257	.870	1.98	.640	.613	2.52	.472	.722
11	2.80	.166	.914	2.20	.536	.709	2.80	.369	.809
12	3.08	.113	.943	2.42	.414	.786	3.08	.247	.873
13	3.36	.078	.963	2.64	.303	.845	3.36	.168	.915
14	3.64	.052	.976	2.86	.219	.887	3.64	.113	.945
15	3.92	.034	.985	3.08	.160	.918	3.92	.075	.964
16	4.20	.023	.991	3.30	.117	.941	4.20	.050	.977
17	4.48	.015	.995	3.52	.088	.947	4.48	.034	.986
18	4.76	.009	.998	3.74	.064	.970	4.76	.021	.991
19	5.04	.004	.999	3.96	.047	.979	5.04	.014	.995
20	5.32	.002	1.000	4.18	.035	.985	5.32	.008	.997
21	5.60	.001	1.000	4.40	.025	.990	5.60	.004	.998
22	5.88	0	1.000	4.62	.018	.994	5.88	.003	.999
23				4.84	.012	.996	6.16	.002	1.000
24				5.06	.007	.998	6.44	.001	1.000
25				5.28	.004	.999	6.72	0	1.000
26				5.50	.003	.999			
27				5.72	.002	1.000			
28				5.94	.001	1.000			
29				6.16	0	1.000			

Table 21.17 (Continued) Hydrograph Family 2

Line No.	$T_o/T_p = 3$ t/T_p	q_c/q_p	Q_t/Q	$T_o/T_p = 4$ t/T_p	q_c/q_p	Q_t/Q	$T_o/T_p = 6$ t/T_p	q_c/q_p	Q_t/Q
1	0	0	0	0	0	0	0	0	0
2	.32	.003	.000	.32	.002	.000	.34	.001	.000
3	.64	.017	.003	.64	.009	.002	.68	.005	.001
4	.96	.093	.016	.96	.036	.007	1.02	.015	.003
5	1.28	.311	.064	1.28	.129	.026	1.36	.037	.010
6	1.60	.530	.163	1.60	.332	.081	1.70	.098	.027
7	1.92	.615	.298	1.92	.501	.179	2.04	.244	.070
8	2.24	.575	.439	2.24	.550	.303	2.38	.407	.151
9	2.56	.487	.565	2.56	.500	.426	2.72	.464	.261
10	2.88	.409	.671	2.88	.422	.535	3.06	.429	.373
11	3.20	.344	.760	3.20	.358	.627	3.40	.367	.473
12	3.52	.279	.834	3.52	.302	.705	3.74	.309	.557
13	3.84	.206	.891	3.84	.274	.773	4.08	.261	.629
14	4.16	.135	.931	4.16	.230	.832	4.42	.224	.690
15	4.48	.087	.958	4.48	.195	.882	4.76	.193	.742
16	4.80	.054	.974	4.80	.147	.922	5.10	.169	.787
17	5.12	.032	.984	5.12	.099	.951	5.44	.152	.828
18	5.44	.019	.990	5.44	.061	.970	5.78	.139	.864
19	5.76	.012	.994	5.76	.037	.982	6.12	.129	.898
20	6.08	.008	.997	6.08	.023	.989	6.46	.113	.928
21	6.40	.005	.998	6.40	.013	.993	6.80	.085	.953
22	6.72	.003	.999	6.72	.008	.996	7.14	.055	.971
23	7.04	.002	1.000	7.04	.005	.997	7.48	.035	.982
24	7.36	.001	1.000	7.36	.004	.998	7.82	.020	.989
25	7.68	0	1.000	7.68	.003	.999	8.16	.012	.993
26				8.00	.002	1.000	8.50	.008	.995
27				8.32	.001	1.000	8.84	.005	.997
28				8.64	0	1.000	9.18	.004	.998
29							9.52	.003	.999
30							9.86	.002	.999
31							10.20	.001	1.000
32							10.54	0	1.000

Table 21.17 (Continued) Hydrograph Family 2

Line No.	$T_o/T_p = 10$ t/T_p	q_c/q_p	Q_t/Q	$T_o/T_p = 16$ t/T_p	q_c/q_p	Q_t/Q	$T_o/T_p = 25$ t/T_p	q_c/q_p	Q_t/Q
1	0	0	0	0	0	0	0	0	0
2	.63	.002	.000	.90	.002	.001	1.30	.002	.001
3	1.26	.009	.003	1.80	.007	.004	2.60	.006	.005
4	1.89	.027	.011	2.70	.020	.013	3.90	.014	.014
5	2.52	.063	.032	3.60	.037	.031	5.20	.024	.032
6	3.15	.236	.102	4.50	.148	.093	6.50	.088	.086
7	3.78	.364	.241	5.40	.277	.233	7.80	.210	.228
8	4.41	.307	.397	6.30	.214	.396	9.10	.146	.397
9	5.04	.226	.521	7.20	.149	.516	10.40	.097	.513
10	5.67	.172	.613	8.10	.112	.603	11.70	.072	.593
11	6.30	.136	.685	9.00	.088	.669	13.00	.057	.655
12	6.93	.113	.743	9.90	.073	.722	14.30	.049	.705
13	7.56	.097	.792	10.80	.063	.767	15.60	.044	.750
14	8.19	.085	.834	11.70	.056	.807	16.90	.039	.789
15	8.82	.078	.872	12.60	.052	.842	18.20	.035	.824
16	9.45	.074	.907	13.50	.048	.875	19.50	.033	.857
17	10.08	.069	.940	14.40	.045	.906	20.80	.031	.887
18	10.71	.053	.969	15.30	.044	.936	22.10	.029	.916
19	11.34	.025	.987	16.20	.042	.964	23.40	.028	.943
20	11.97	.009	.995	17.10	.023	.986	24.70	.027	.969
21	12.60	.004	.998	18.00	.006	.995	26.00	.014	.989
22	13.23	.002	.999	18.90	.003	.998	27.30	.004	.997
23	13.86	.001	1.000	19.80	.001	1.000	28.60	.001	1.000
24	14.49	0	1.000	20.70	0	1.000	29.90	0	1.000

Table 21.17 (Continued) Hydrograph Family 2

Line No.	$T_o/T_p = 36$			$T_o/T_p = 50$			$T_o/T_p = 75$		
	t/T_p	q_c/q_p	Q_t/Q	t/T_p	q_c/q_p	Q_t/Q	t/T_p	q_c/q_p	Q_t/Q
1	0	0	0	0	0	0	0	0	0
2	1.79	.002	.001	2.50	.0018	.002	3.00	.0012	.001
3	3.58	.006	.007	5.00	.0047	.008	6.00	.0027	.006
4	5.37	.012	.019	7.50	.0087	.020	9.00	.0044	.014
5	7.16	.019	.039	10.00	.0145	.041	12.00	.0067	.026
6	8.95	.057	.909	12.50	.0615	.111	15.00	.0108	.045
7	10.74	.157	.232	15.00	.1184	.276	18.00	.0309	.091
8	12.53	.104	.405	17.50	.0621	.442	21.00	.0790	.213
9	14.32	.068	.519	20.00	.0433	.539	24.00	.0624	.369
10	16.11	.047	.596	22.50	.0342	.611	27.00	.0357	.478
11	17.90	.040	.653	25.00	.0274	.667	30.00	.0283	.548
12	19.69	.034	.703	27.50	.0234	.714	33.00	.0234	.606
13	21.48	.030	.745	30.00	.0209	.755	36.00	.0196	.653
14	23.27	.026	.782	32.50	.0187	.791	39.00	.0167	.693
15	25.06	.025	.816	35.00	.0167	.824	42.00	.0150	.728
16	26.85	.023	.848	37.50	.0159	.854	45.00	.0137	.760
17	28.64	.021	.877	40.00	.0153	.882	48.00	.0126	.789
18	30.43	.020	.904	42.50	.0147	.910	51.00	.0115	.816
19	32.22	.019	.930	45.00	.0142	.936	54.00	.0108	.840
20	34.01	.018	.955	47.50	.0136	.962	57.00	.0104	.864
21	35.80	.017	.978	50.00	.0131	.986	60.00	.0101	.886
22	37.59	.007	.994	52.50	.0008	.999	63.00	.0098	.908
23	39.38	.001	.999	55.00	0	1.000	66.00	.0095	.930
24	41.17	0	1.000				69.00	.0092	.950
25							72.00	.0089	.970
26							75.00	.0086	.990
27							78.00	.0003	1.000
28							81.00	0	1.000

Table 21.17 (Continued)

Hydrograph Family 3

Line No.	$T_o/T_p = 1$ t/T_p	q_c/q_p	Q_t/Q	$T_o/T_p = 1.5$ t/T_p	q_c/q_p	Q_t/Q	$T_o/T_p = 2$ t/T_p	q_c/q_p	Q_t/Q
1	0	0	0	0	0	0	0	0	0
2	.26	.048	.005	.29	.028	.003	.30	.012	.001
3	.52	.219	.030	.58	.190	.026	.60	.123	.016
4	.78	.521	.101	.87	.450	.094	.90	.343	.068
5	1.04	.762	.224	1.16	.656	.212	1.20	.570	.169
6	1.30	.844	.378	1.45	.734	.360	1.50	.657	.304
7	1.56	.778	.533	1.74	.685	.511	1.80	.630	.447
8	1.82	.621	.668	2.03	.585	.646	2.10	.562	.578
9	2.08	.441	.769	2.32	.445	.756	2.40	.484	.694
10	2.34	.305	.841	2.61	.350	.841	2.70	.379	.789
11	2.60	.214	.891	2.90	.199	.899	3.00	.267	.861
12	2.86	.149	.925	3.19	.132	.934	3.30	.177	.910
13	3.12	.103	.949	3.48	.089	.958	3.60	.116	.942
14	3.38	.070	.966	3.77	.057	.973	3.90	.076	.964
15	3.64	.048	.977	4.06	.038	.983	4.20	.050	.977
16	3.90	.034	.985	4.35	.025	.990	4.50	.033	.987
17	4.16	.024	.991	4.64	.015	.994	4.80	.020	.992
18	4.42	.016	.995	4.93	.008	.997	5.10	.011	.996
19	4.68	.010	.997	5.22	.005	.998	5.40	.006	.998
20	4.94	.006	.999	5.51	.003	.999	5.70	.004	.999
21	5.20	.003	1.000	5.80	.002	1.000	6.00	.002	1.000
22	5.46	.001	1.000	6.09	.001	1.000	6.30	.001	1.000
23	5.72	0	1.000	6.38	0	1.000	6.60	0	1.000

Table 21.17 (Continued) Hydrograph Family 3

Line No.	$T_o/T_p = 3$ t/T_p	q_c/q_p	Q_t/Q	$T_o/T_p = 4$ t/T_p	q_c/q_p	Q_t/Q	$T_o/T_p = 6$ t/T_p	q_c/q_p	Q_t/Q
1	0	0	0	0	0	0	0	0	0
2	.34	.004	.001	.36	.003	.000	.42	.002	.000
3	.68	.088	.012	.72	.044	.007	.84	.021	.004
4	1.02	.289	.059	1.08	.203	.040	1.26	.138	.029
5	1.36	.489	.157	1.44	.400	.120	1.68	.320	.100
6	1.70	.543	.286	1.80	.478	.237	2.10	.390	.210
7	2.04	.507	.418	2.16	.450	.360	2.52	.363	.327
8	2.38	.445	.537	2.52	.397	.473	2.94	.314	.432
9	2.72	.385	.641	2.88	.342	.572	3.36	.270	.522
10	3.06	.340	.732	3.24	.296	.656	3.78	.232	.600
11	3.40	.294	.811	3.60	.257	.730	4.20	.199	.667
12	3.74	.223	.876	3.96	.234	.795	4.62	.174	.725
13	4.08	.149	.922	4.32	.210	.855	5.04	.155	.776
14	4.42	.096	.953	4.68	.169	.905	5.46	.144	.822
15	4.76	.056	.972	5.04	.111	.942	5.88	.137	.866
16	5.10	.033	.983	5.40	.067	.966	6.30	.127	.907
17	5.44	.019	.990	5.76	.037	.980	6.72	.101	.942
18	5.78	.013	.994	6.12	.022	.988	7.14	.063	.968
19	6.12	.008	.996	6.48	.014	.993	7.56	.033	.983
20	6.46	.004	.998	6.84	.008	.995	7.98	.018	.991
21	6.80	.003	.999	7.20	.006	.997	8.40	.010	.995
22	7.14	.002	.999	7.56	.004	.999	8.82	.005	.997
23	7.48	.001	1.000	7.92	.002	.999	9.24	.003	.998
24	7.82	0	1.000	8.28	.001	1.000	9.66	.002	.999
25				8.64	0	1.000	10.08	.001	1.000
26							10.50		1.000
27							10.92	0	1.000

Table 21.17(Continued) Hydrograph Family 3

	$T_o/T_p = 10$			$T_o/T_p = 16$			$T_o/T_p = 25$		
Line No.	t/T_p	q_c/q_p	Q_t/Q	t/T_p	q_c/q_p	Q_t/Q	t/T_p	q_c/q_p	Q_t/Q
1	0	0	0	0	0	0	0	0	0
2	.54	.001	.000	.90	.002	.001	1.23	.002	.001
3	1.08	.008	.002	1.80	.016	.007	2.46	.009	.006
4	1.62	.069	.017	2.70	.122	.053	3.69	.073	.043
5	2.16	.231	.077	3.60	.230	.170	4.92	.173	.154
6	2.70	.303	.184	4.50	.185	.308	6.15	.132	.291
7	3.24	.269	.298	5.40	.139	.415	7.38	.096	.394
8	3.78	.223	.396	6.30	.113	.499	8.61	.076	.471
9	4.32	.188	.478	7.20	.094	.568	9.84	.064	.534
10	4.86	.159	.548	8.10	.081	.626	11.07	.055	.588
11	5.40	.139	.607	9.00	.072	.677	12.30	.050	.635
12	5.94	.122	.659	9.90	.064	.722	13.53	.046	.678
13	6.48	.108	.705	10.80	.057	.762	14.76	.042	.718
14	7.02	.097	.746	11.70	.053	.799	15.99	.038	.754
15	7.56	.089	.783	12.60	.050	.833	17.22	.035	.787
16	8.10	.081	.817	13.50	.049	.866	18.45	.033	.818
17	8.64	.078	.849	14.40	.048	.898	19.68	.032	.947
18	9.18	.077	.880	15.30	.047	.930	20.91	.031	.875
19	9.72	.077	.911	16.20	.046	.961	22.14	.031	.903
20	10.26	.075	.941	17.10	.024	.984	23.37	.031	.931
21	10.80	.055	.967	18.00	.006	.994	24.60	.031	.959
22	11.34	.030	.984	18.90	.004	.997	25.83	.025	.984
23	11.88	.012	.992	19.80	.002	.999	27.06	.004	.997
24	12.42	.006	.996	20.70	0	1.000	28.29	.001	1.000
25	12.96	.004	.998				29.52	0	1.000
26	13.50	.002	.999						
27	14.04	.001	1.000						
28	14.58	0	1.000						

Table 21.17 (Continued) Hydrograph Family 3

Line No.	$T_o/T_p = 36$ t/T_p	q_c/q_p	Q_t/Q	$T_o/T_p = 50$ t/T_p	q_c/q_p	Q_t/Q	$T_o/T_p = 75$ t/T_p	q_c/q_p	Q_t/Q
1	0	0	0	0	0	0	0	0	0
2	1.62	.002	.001	2.25	.0008	.001	3.25	.0009	.001
3	3.24	.006	.006	4.50	.0070	.007	6.50	.0057	.009
4	4.86	.047	.037	6.75	.0474	.052	9.75	.0289	.051
5	6.48	.130	.143	9.00	.0972	.173	13.00	.0667	.166
6	8.10	.097	.277	11.25	.0642	.307	16.25	.0445	.299
7	9.72	.069	.376	13.50	.0460	.399	19.50	.0317	.391
8	11.34	.052	.448	15.75	.0375	.469	22.75	.0257	.460
9	12.96	.045	.505	18.00	.0322	.527	26.00	.0219	.517
10	14.58	.041	.551	20.25	.0285	.577	29.25	.0195	.567
11	16.20	.037	.603	22.50	.0258	.622	32.50	.0176	.612
12	17.82	.034	.645	24.75	.0239	.664	35.75	.0160	.652
13	19.44	.031	.683	27.00	.0219	.702	39.00	.0147	.689
14	21.06	.028	.719	29.25	.0201	.737	42.25	.0136	.723
15	22.68	.025	.750	31.50	.0185	.769	45.50	.0127	.755
16	24.30	.024	.779	33.75	.0173	.799	48.75	.0118	.784
17	25.92	.024	.808	36.00	.0165	.829	52.00	.0113	.812
18	27.54	.024	.836	38.25	.0162	.854	55.25	.0109	.839
19	29.16	.024	.865	40.50	.0159	.881	58.50	.0107	.865
20	30.78	.023	.893	42.75	.0156	.907	61.75	.0105	.890
21	32.40	.023	.920	45.00	.0153	.933	65.00	.0103	.915
22	34.02	.023	.947	47.25	.0150	.958	68.25	.0101	.940
23	35.64	.023	.974	49.50	.0147	.983	71.50	.0099	.964
24	37.26	.007	.992	51.75	.0028	.998	74.75	.0097	.988
25	38.88	.003	.998	54.00	0	1.000	78.00	.0003	1.000
26	40.50	0	1.000				81.25	0	1.000

Table 21.17 (Continued)

Hydrograph Family 4

Line No.	$T_o/T_p = 1$ t/T_p	q_c/q_p	Q_t/Q	$T_o/T_p = 1.5$ t/T_p	q_c/q_p	Q_t/Q	$T_o/T_p = 2$ t/T_p	q_c/q_p	Q_t/Q
1	0	0	0	0	0	0	0	0	0
2	.28	.051	.005	.28	.038	.004	.32	.031	.004
3	.56	.220	.033	.56	.166	.025	.64	.173	.028
4	.84	.490	.107	.84	.360	.079	.96	.360	.091
5	1.12	.738	.234	1.12	.551	.174	1.28	.494	.191
6	1.40	.830	.397	1.40	.651	.298	1.60	.555	.315
7	1.68	.751	.560	1.68	.686	.436	1.92	.567	.447
8	1.96	.573	.697	1.96	.650	.575	2.24	.555	.580
9	2.24	.392	.797	2.24	.543	.698	2.56	.490	.703
10	2.52	.259	.865	2.52	.392	.795	2.88	.370	.805
11	2.80	.174	.910	2.80	.267	.863	3.20	.242	.877
12	3.08	.118	.940	3.08	.180	.909	3.52	.150	.923
13	3.36	.079	.960	3.36	.120	.940	3.84	.098	.952
14	3.64	.053	.974	3.64	.081	.961	4.16	.063	.971
15	3.92	.036	.983	3.92	.055	.975	4.48	.038	.983
16	4.20	.025	.990	4.20	.036	.984	4.80	.024	.991
17	4.48	.017	.994	4.48	.024	.991	5.12	.013	.995
18	4.76	.011	.997	4.76	.015	.995	5.44	.008	.997
19	5.04	.006	.999	5.04	.009	.997	5.76	.004	.999
20	5.32	.003	.999	5.32	.005	.999	6.08	.002	.999
21	5.60	.001	1.000	5.60	.003	.999	6.40	.001	1.000
22	5.88	0	1.000	5.88	.001	1.000	6.72	0	1.000
23				6.16	0	1.000			

Table 21.17 (Continued) Hydrograph Family 4

Line No.	$T_o/T_p = 3$ t/T_p	q_c/q_p	Qt/Q	$T_o/T_p = 4$ t/T_p	q_c/q_p	Qt/Q	$T_o/T_p = 6$ t/T_p	q_c/q_p	Qt/Q
1	0	0	0	0	0	0	0	0	0
2	.28	.018	.002	.40	.023	. 03	.40	.014	.002
3	.56	.086	.013	.80	.143	.028	.80	.088	.017
4	.84	.200	.042	1.20	.272	.089	1.20	.191	.058
5	1.12	.311	.095	1.60	.326	.177	1.60	.244	.122
6	1.40	.386	.167	2.00	.340	.276	2.00	.250	.195
7	1.68	.415	.250	2.40	.337	.376	2.40	.246	.268
8	1.96	.422	.337	2.80	.323	.473	2.80	.240	.340
9	2.24	.417	.424	3.20	.306	.566	3.20	.233	.410
10	2.52	.402	.509	3.60	.293	.654	3.60	.223	.477
11	2.80	.394	.591	4.00	.286	.740	4.00	.212	.541
12	3.08	.387	.672	4.40	.266	.821	4.40	.202	.602
13	3.36	.363	.750	4.80	.197	.890	4.80	.194	.660
14	3.64	.316	.820	5.20	.122	.937	5.20	.189	.717
15	3.92	.236	.877	5.60	.067	.965	5.60	.187	.772
16	4.20	.164	.919	6.00	.036	.980	6.00	.185	.827
17	4.48	.108	.947	6.40	.021	.988	6.40	.175	.880
18	4.76	.073	.966	6.80	.013	.993	6.80	.131	.925
19	5.04	.047	.978	7.20	.008	.996	7.20	.080	.956
20	5.32	.030	.986	7.60	.005	.998	7.60	.046	.975
21	5.60	.020	.991	8.00	.002	.999	8.00	.027	.985
22	5.88	.013	.995	8.40	.001	1.000	8.40	.016	.992
23	6.16	.008	.997	8.80	0	1.000	8.80	.009	.995
24	6.44	.005	.998				9.20	.005	.997
25	6.72	.003	.999				9.60	.003	.999
26	7.00	.002	1.000				10.00	.002	.999
27	7.28	.001	1.000				10.40	.001	1.000
28	7.56		1.000				10.80	0	1.000
29	7.84	0	1.000						

Table 21.17 (Continued) Hydrograph Family 4

	$T_o/T_p = 10$			$T_o/T_p = 16$			$T_o/T_p = 25$		
Line No.	t/T_p	q_c/q_p	Q_t/Q	t/T_p	q_c/q_p	Q_t/Q	t/T_p	q_c/q_p	Q_t/Q
1	0	0	0	0	0	0	0	0	0
2	.50	.015	.003	.62	.015	.003	1.02	.025	.009
3	1.00	.079	.020	1.24	.064	.022	2.04	.070	.045
4	1.50	.151	.062	1.86	.112	.062	3.06	.092	.106
5	2.00	.177	.122	2.48	.128	.117	4.08	.082	.170
6	2.50	.170	.186	3.10	.119	.173	5.10	.068	.227
7	3.00	.159	.247	3.72	.105	.225	6.12	.062	.276
8	3.50	.152	.304	4.34	.097	.271	7.14	.059	.321
9	4.00	.146	.358	4.96	.094	.315	8.16	.056	.365
10	4.50	.141	.411	5.58	.091	.357	9.18	.055	.407
11	5.00	.136	.462	6.20	.089	.398	10.20	.054	.448
12	5.50	.131	.511	6.82	.087	.438	11.22	.053	.488
13	6.00	.126	.558	7.44	.085	.478	12.24	.052	.528
14	6.50	.121	.604	8.06	.082	.516	13.26	.050	.566
15	7.00	.116	.647	8.68	.079	.553	14.28	.049	.603
16	7.50	.112	.689	9.30	.076	.588	15.30	.047	.639
17	8.00	.112	.730	9.92	.074	.623	16.32	.046	.674
18	8.50	.111	.771	10.54	.072	.656	17.34	.045	.709
19	9.00	.111	.812	11.16	.071	.689	18.36	.044	.742
20	9.50	.110	.852	11.78	.070	.721	19.38	.044	.775
21	10.00	.110	.893	12.40	.069	.753	20.40	.044	.809
22	10.50	.100	.931	13.02	.069	.785	21.42	.044	.842
23	11.00	.065	.962	13.64	.069	.816	22.44	.044	.875
24	11.50	.033	.980	14.26	.069	.848	23.46	.044	.908
25	12.00	.025	.990	14.88	.069	.879	24.48	.044	.941
26	12.50	.007	.996	15.50	.069	.911	25.50	.039	.972
27	13.00	.004	.998	16.12	.068	.942	26.52	.012	.992
28	13.50	.002	.999	16.74	.053	.970	27.54	.004	.998
29	14.00	.001	1.000	17.36	.023	.987	28.56	.001	1.000
30	14.50	0	1.000	17.98	.009	.995	29.58	0	1.000
31				18.60	.004	.998			
32				19.22	.002	.999			
33				19.84	.001	1.000			
34				20.46	0	1.000			

Table 21.17 (Continued)

	Hydrograph Family 4						Hydrograph Family 5		
	$T_o/T_p = 36$			$T_o/T_p = 50$			$T_o/T_p = 1$		
Line No.	t/T_p	q_c/q_p	Q_t/Q	t/T_p	q_c/q_p	Q_t/Q	t/T_p	q_c/q_p	Q_t/Q
1	0	0	0	0	0	0	0	0	0
2	1.50	.0306	.017	2.00	.0277	.020	.26	.021	.002
3	3.00	.0575	.066	4.00	.0464	.075	.52	.106	.014
4	4.50	.0672	.135	6.00	.0435	.141	.78	.289	.052
5	6.00	.0492	.199	8.00	.0378	.201	1.04	.530	.131
6	7.50	.0433	.251	10.00	.0335	.254	1.30	.740	.254
7	9.00	.0418	.298	12.00	.0307	.301	1.56	.848	.407
8	10.50	.0408	.344	14.00	.0291	.345	1.82	.767	.563
9	12.00	.0400	.388	16.00	.0282	.388	2.08	.590	.693
10	13.50	.0391	.432	18.00	.0274	.429	2.34	.406	.789
11	15.00	.0382	.475	20.00	.0266	.468	2.60	.279	.855
12	16.50	.0371	.517	22.00	.0258	.507	2.86	.193	.901
13	18.00	.0358	.557	24.00	.0250	.544	3.12	.134	.933
14	19.50	.0341	.596	26.00	.0242	.581	3.38	.092	.954
15	21.00	.0319	.632	28.00	.0234	.616	3.64	.065	.969
16	22.50	.0308	.667	30.00	.0230	.650	3.90	.044	.980
17	24.00	.0306	.701	32.00	.0229	.683	4.16	.030	.987
18	25.50	.0306	.735	34.00	.0227	.718	4.42	.021	.992
19	27.00	.0306	.769	36.00	.0226	.751	4.68	.015	.995
20	28.50	.0306	.803	38.00	.0225	.784	4.94	.009	.998
21	30.00	.0306	.837	40.00	.0224	.817	5.20	.005	.999
22	31.50	.0306	.871	42.00	.0222	.850	5.46	.002	1.000
23	33.00	.0306	.905	44.00	.0221	.883	5.72	0	1.000
24	34.50	.0306	.939	46.00	.0219	.915			
25	36.00	.0306	.973	48.00	.0219	.948			
26	37.50	.0085	.994	50.00	.0217	.980			
27	39.00	.0009	1.000	52.00	.0029	.998			
28	40.50	0	1.000	54.00	0	1.000			

Table 21.17 (Continued) Hydrograph Family 5

Line No.	$T_o/T_p = 1.5$			$T_o/T_p = 2$			$T_o/T_p = 3$		
	t/T_p	q_c/q_p	Q_t/Q	t/T_p	q_c/q_p	Q_t/Q	t/T_p	q_c/q_p	Q_t/Q
1	0	0	0	0	0	0	0	0	0
2	.25	.013	.001	.25	.010	.001	.34	.010	.001
3	.50	.065	.008	.50	.048	.006	.68	.068	.011
4	.75	.173	.030	.75	.127	.022	1.02	.150	.039
5	1.00	.306	.075	1.00	.227	.055	1.36	.229	.086
6	1.25	.434	.143	1.25	.318	.106	1.70	.283	.151
7	1.50	.562	.235	1.50	.389	.171	2.04	.315	.226
8	1.75	.680	.350	1.75	.448	.248	2.38	.339	.308
9	2.00	.737	.481	2.00	.523	.338	2.72	.378	.399
10	2.25	.673	.611	2.25	.609	.443	3.06	.459	.504
11	2.50	.530	.722	2.50	.642	.558	3.40	.509	.626
12	3.75	.381	.806	2.75	.576	.671	3.74	.446	.746
13	3.00	.262	.866	3.00	.450	.766	4.08	.310	.841
14	3.25	.185	.907	3.25	.322	.837	4.42	.190	.904
15	3.50	.129	.936	3.50	.222	.888	4.76	.117	.943
16	3.75	.090	.956	3.75	.156	.923	5.10	.069	.966
17	4.00	.063	.970	4.00	.109	.947	5.44	.040	.980
18	4.25	.045	.980	4.25	.075	.964	5.78	.025	.988
19	4.50	.031	.987	4.50	.053	.976	6.12	.016	.993
20	4.75	.022	.992	4.75	.037	.984	6.46	.009	.997
21	5.00	.014	.995	5.00	.025	.990	6.80	.005	.998
22	5.25	.009	.998	5.25	.017	.994	7.14	.003	.999
23	5.50	.005	.999	5.50	.011	.996	7.48	.001	1.000
24	5.75	.003	1.000	5.75	.007	.998	7.82	0	1.000
25	6.00	.001	1.000	6.00	.004	.999			
26	6.25	0	1.000	6.25	.002	1.000			
27				6.50	.001	1.000			
28				6.75	0	1.000			

Table 21.17 (Continued) Hydrograph Family 5

Line No.	$T_o/T_p = 4$ t/T_p	q_c/q_p	Q_t/Q	$T_o/T_p = 6$ t/T_p	q_c/q_p	Q_t/Q	$T_o/T_p = 10$ t/T_p	q_c/q_p	Q_t/Q
1	0	0	0	0	0	0	0	0	0
2	.36	.010	.001	.52	.015	.003	.67	.013	.003
3	.72	.053	.010	1.04	.070	.019	1.34	.061	.022
4	1.08	.124	.033	1.56	.130	.057	2.01	.091	.059
5	1.44	.181	.074	2.08	.159	.112	2.68	.102	.107
6	1.80	.220	.127	2.60	.172	.176	3.35	.107	.159
7	2.16	.243	.189	3.12	.178	.242	4.02	.110	.213
8	2.52	.256	.255	3.64	.182	.311	4.69	.111	.268
9	2.88	.263	.325	4.16	.183	.381	5.36	.111	.323
10	3.24	.273	.396	4.68	.184	.451	6.03	.112	.378
11	3.60	.308	.473	5.20	.218	.527	6.70	.112	.434
12	3.96	.380	.565	5.72	.285	.623	7.37	.112	.490
13	4.32	.427	.672	6.24	.324	.740	8.04	.116	.546
14	4.68	.377	.779	6.76	.267	.852	8.71	.160	.615
15	5.04	.260	.864	7.28	.133	.929	9.38	.198	.704
16	5.40	.155	.919	7.80	.064	.966	10.05	.212	.805
17	5.76	.094	.953	8.32	.029	.984	10.72	.168	.900
18	6.12	.055	.972	8.84	.016	.993	11.39	.074	.960
19	6.48	.032	.984	9.36	.007	.997	12.06	.027	.985
20	6.84	.019	.991	9.88	.003	.999	12.73	.010	.994
21	7.20	.012	.995	10.40	.001	1.000	13.40	.005	.998
22	7.56	.007	.997	10.92	0	1.000	14.07	.002	1.000
23	7.92	.004	.999				14.74	0	1.000
24	8.28	.002	1.000						
25	8.64	0	1.000						

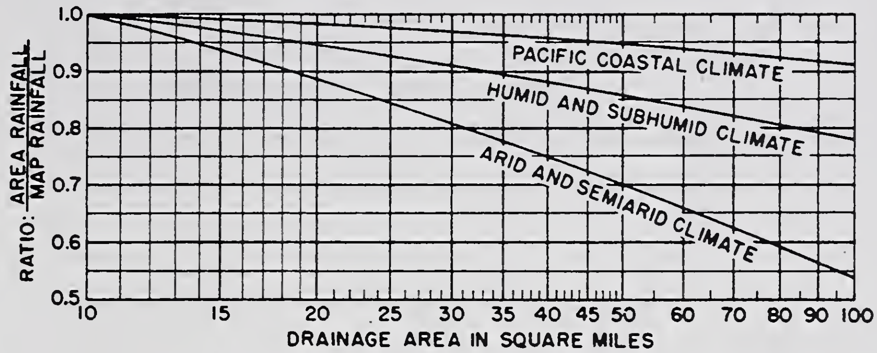
Table 21.17 (Continued) Hydrograph Family 5

	$T_o/T_p = 16$			$T_o/T_p = 25$		
Line No.	t/T_p	q_c/q_p	Qt/Q	t/T_p	q_c/q_p	Qt/Q
1	0	0	0	0	0	0
2	.80	.008	.002	1.25	.015	.007
3	1.60	.046	.018	2.50	.039	.032
4	2.40	.060	.050	3.75	.043	.070
5	3.20	.065	.087	5.00	.044	.110
6	4.00	.067	.126	6.25	.044	.151
7	4.80	.067	.166	7.50	.044	.191
8	5.60	.068	.206	8.75	.044	.232
9	6.40	.068	.246	10.00	.044	.273
10	7.20	.068	.286	11.25	.044	.314
11	8.00	.068	.327	12.50	.044	.354
12	8.80	.068	.367	13.75	.044	.395
13	9.60	.068	.407	15.00	.044	.436
14	10.40	.068	.448	16.25	.044	.476
15	11.20	.068	.488	17.50	.044	.517
16	12.00	.068	.528	18.75	.045	.558
17	12.80	.086	.574	20.00	.067	.610
18	13.60	.121	.636	21.25	.083	.679
19	14.40	.133	.711	22.50	.087	.758
20	15.20	.136	.791	23.75	.087	.839
21	16.00	.137	.872	25.00	.088	.920
22	16.80	.098	.941	26.25	.035	.976
23	17.60	.033	.980	27.50	.006	.995
24	18.40	.012	.993	28.75	.002	.999
25	19.20	.004	.998	30.00	0	1.000
26	20.00	.001	1.000			
27	20.80	0	1.000			

Table 21.17 (Concluded) Hydrograph Family 5

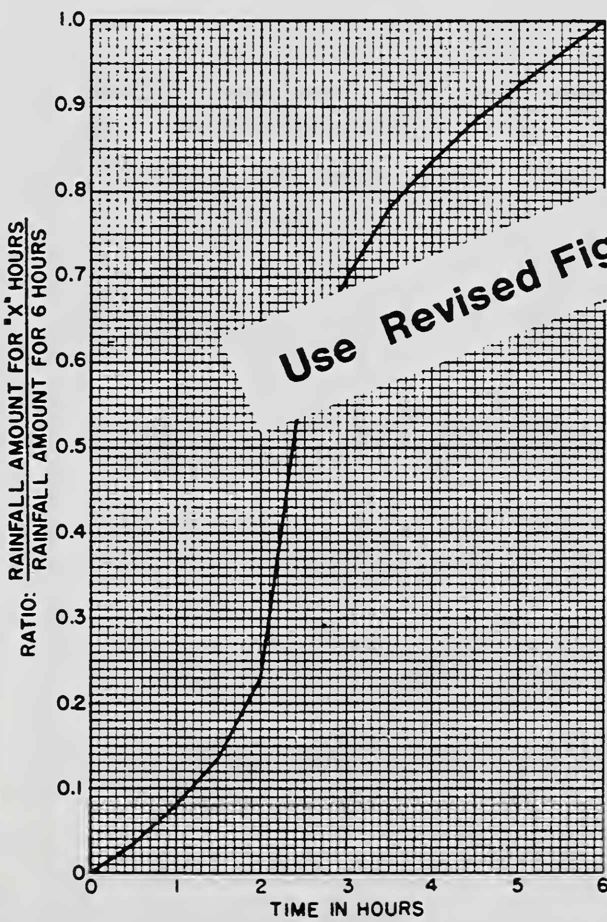
Line No.	$T_o/T_p = 36$ t/Tp	qc/qp	Qt/Q	$T_o/T_p = 50$ t/Tp	Qc/qp	Qt/Q
1	0	0	0	0	0	0
2	1.50	.0195	.011	2.00	.0167	.012
3	3.00	.0275	.037	4.00	.0204	.040
4	4.50	.0294	.068	6.00	.0214	.071
5	6.00	.0300	.101	8.00	.0216	.102
6	7.50	.0301	.135	10.00	.0216	.134
7	9.00	.0301	..168	12.00	.0216	.166
8	10.50	.0301	.202	14.00	.0216	.198
9	12.00	.0301	.235	16.00	.0216	.230
10	13.50	.0301	.268	18.00	.0216	.262
11	15.00	.0301	.302	20.00	.0216	.294
12	16.50	.0301	.335	22.00	.0216	.326
13	18.00	.0301	.369	24.00	.0216	.358
14	19.50	.0301	.402	26.00	.0216	.390
15	21.00	.0301	.435	28.00	.0216	.422
16	22.50	.0301	.469	30.00	.0216	.454
17	24.00	.0311	.503	32.00	.0217	.486
18	25.50	.0364	.540	34.00	.0243	.520
19	27.00	.0425	.584	36.00	.0287	.559
20	28.50	.0480	.634	38.00	.0329	.604
21	30.00	.0525	.690	40.00	.0363	.656
22	31.50	.0561	.750	42.00	.0391	.711
23	33.00	.0584	.814	44.00	.0411	.771
24	34.50	.0598	.879	46.00	.0423	.832
25	36.00	.0603	.946	48.00	.0430	.895
26	37.50	.0167	.989	50.00	.0433	.959
27	39.00	.0018	.999	52.00	.0058	.995
28	40.50	0	1.000	54.00	.0002	1.000
29				56.00	0	1.000

HYDROLOGY: CRITERIA FOR DESIGN STORMS USED IN DEVELOPING EMERGENCY SPILLWAY DESIGN AND FREEBOARD HYDROGRAPHS

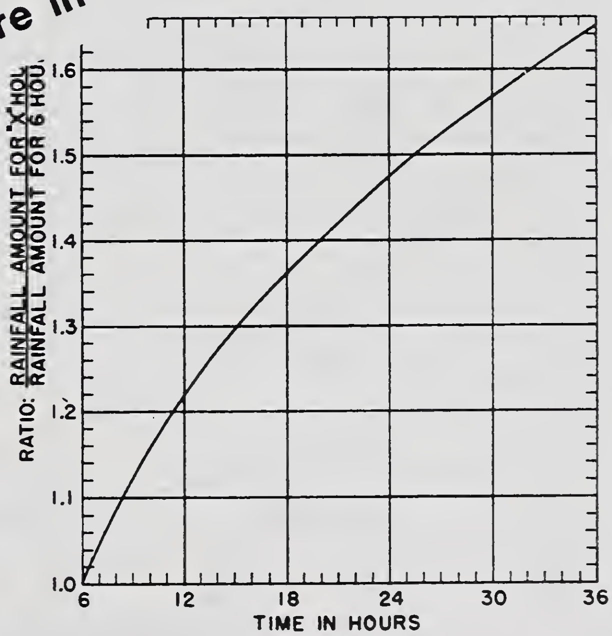


(a) RAINFALL RATIOS FOR DRAINAGE AREAS OF 10 TO 100 SQUARE MILES

Use Revised Figure in TR-60



(b) SIX HOUR DESIGN STORM DISTRIBUTION



(c) RELATIVE INCREASE IN RAINFALL AMOUNT FOR STORM DURATIONS OVER SIX HOURS

FIGURE 21.2

REFERENCE	U. S. DEPARTMENT OF AGRICULTURE SOIL CONSERVATION SERVICE ENGINEERING DIVISION - CENTRAL TECHNICAL UNIT	STANDARD DWG. NO. ES- 1003 SHEET 1 OF 1 DATE 7-2-56 REVISED 9-10-63

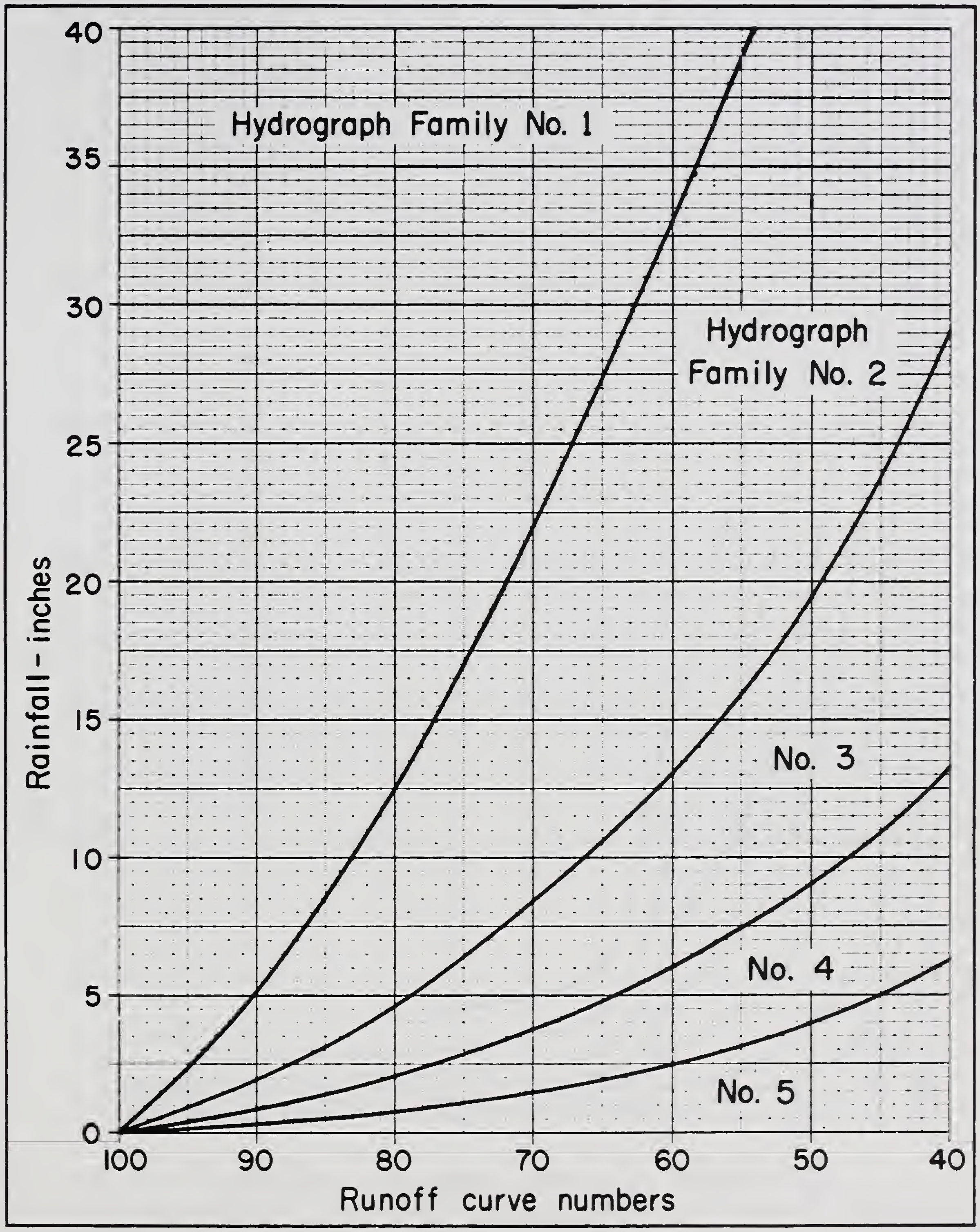


Figure 21-3. Chart for selecting a hydrograph family for a given rainfall and runoff curve number.

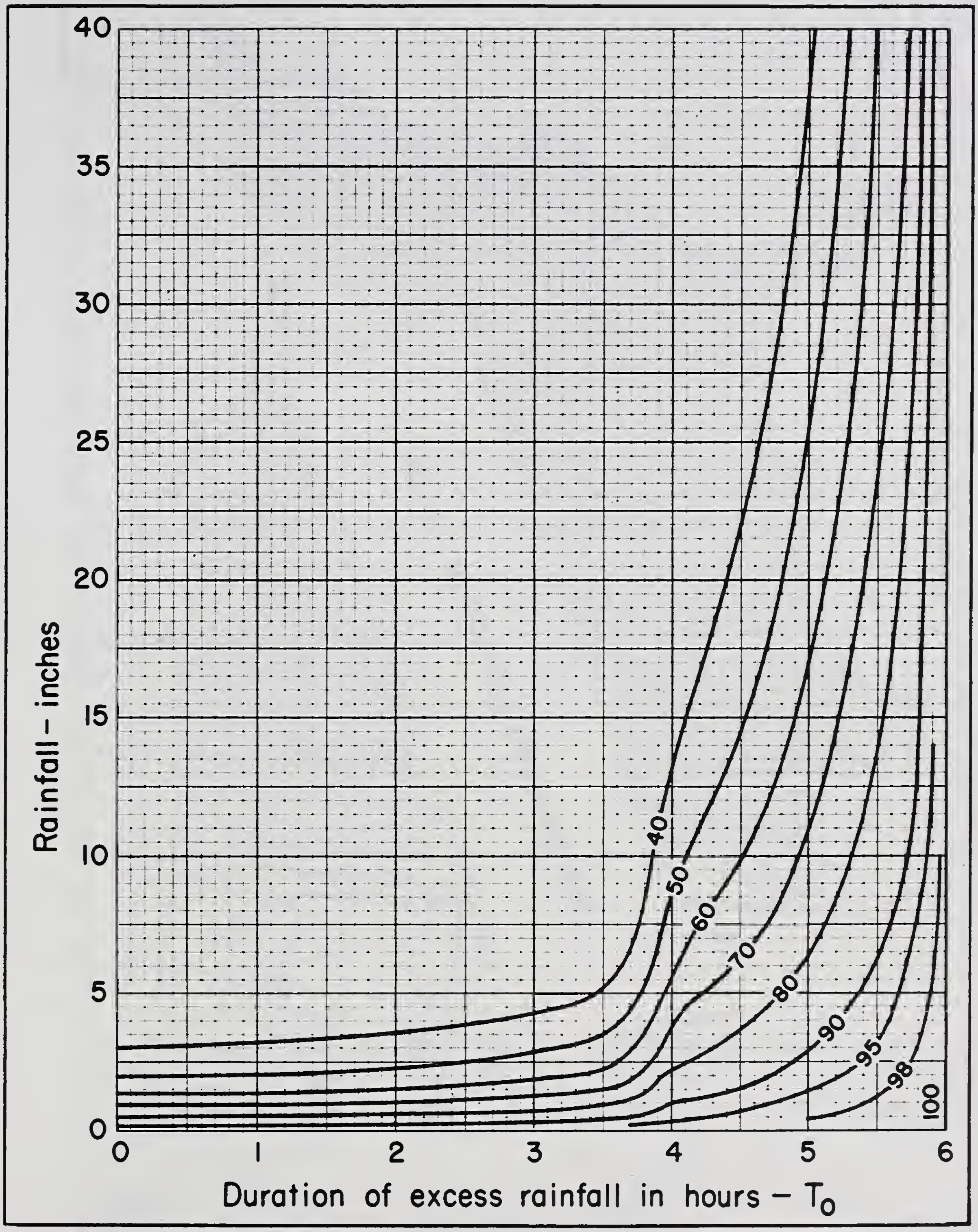


Figure 21-4. Duration of excess rainfall for a 6-hour rainfall and for runoff curve numbers 40 to 100.

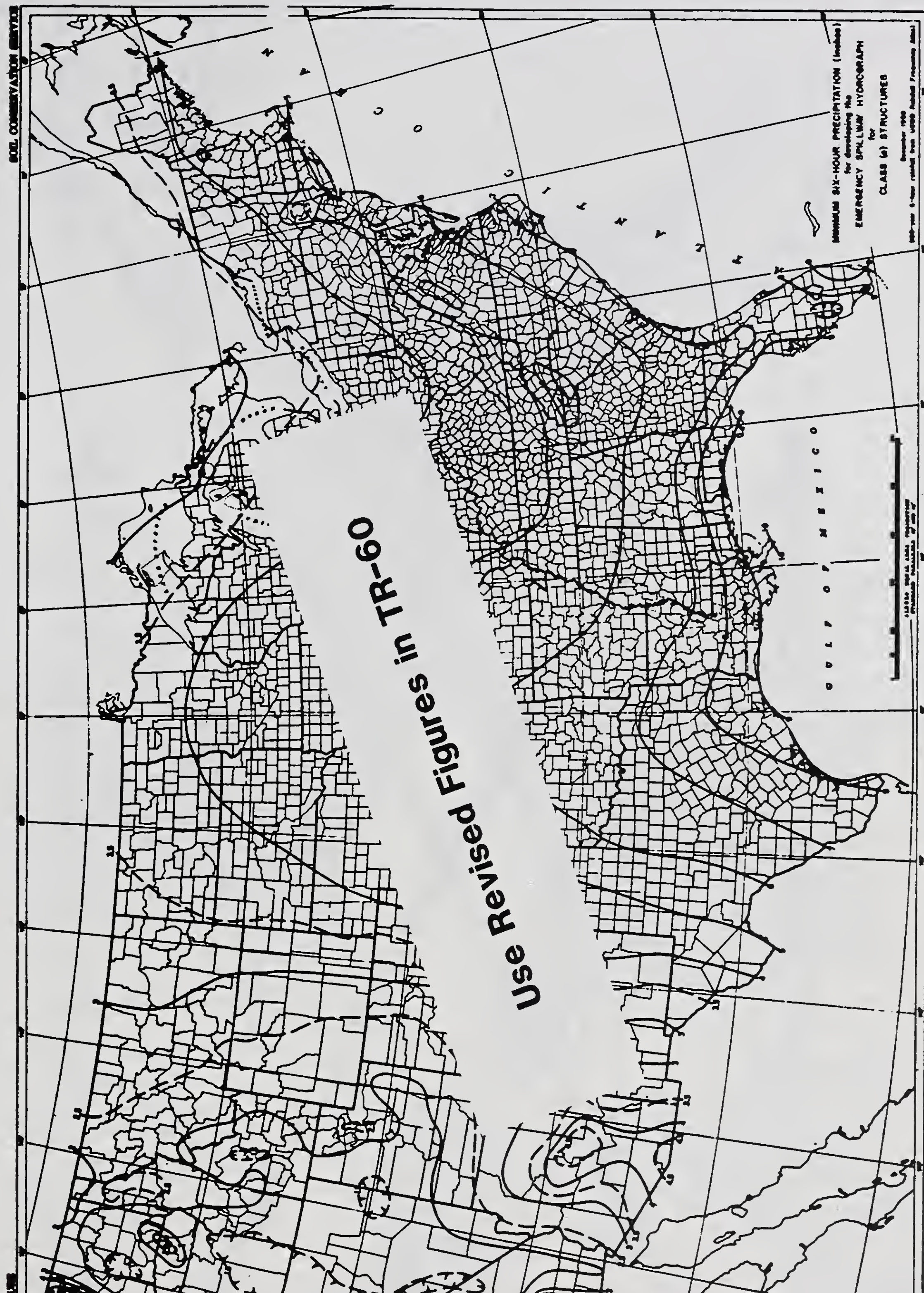


FIGURE 21.5 (1 of 5)

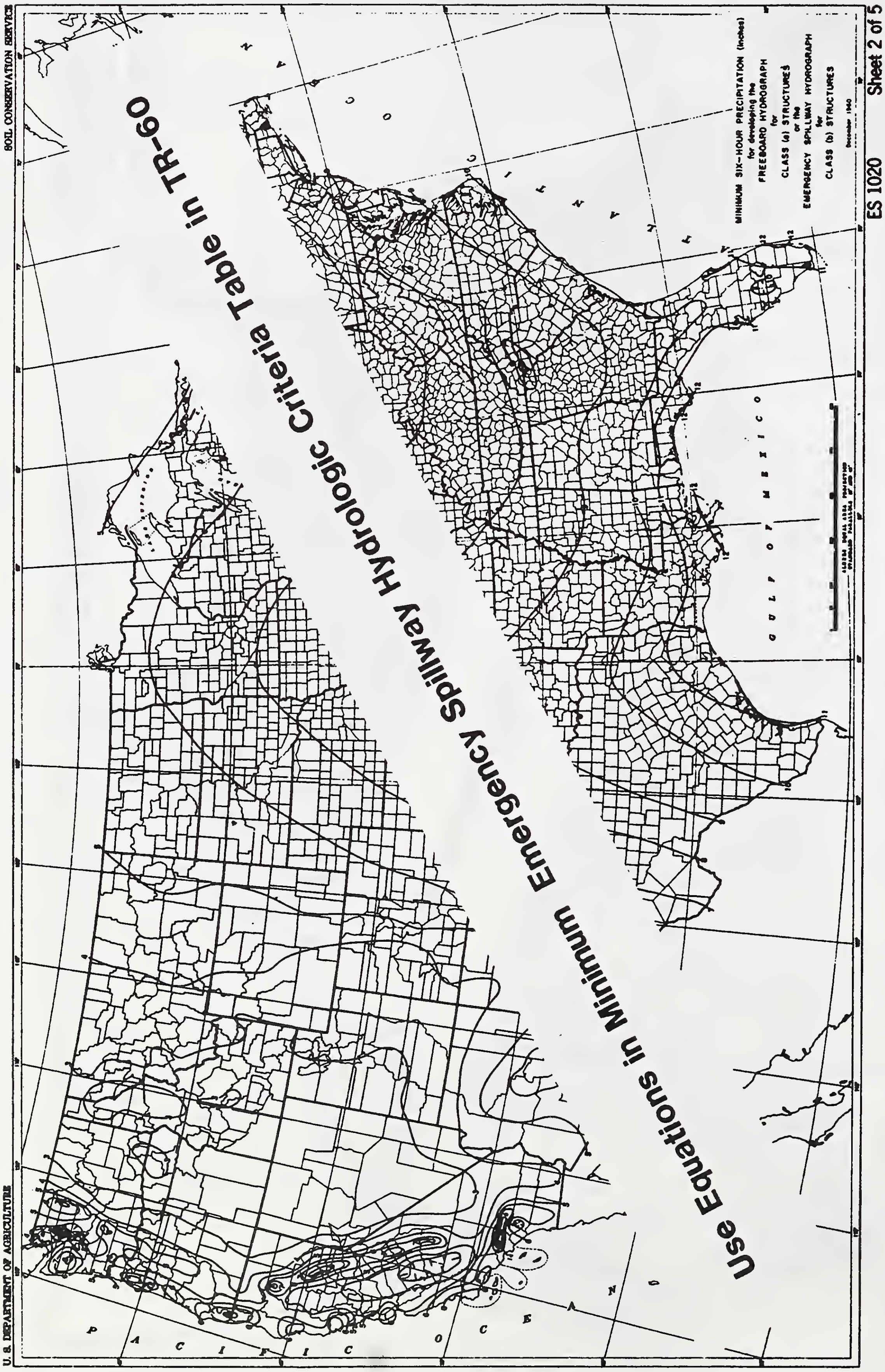


FIGURE 21.5 (2 of 5)

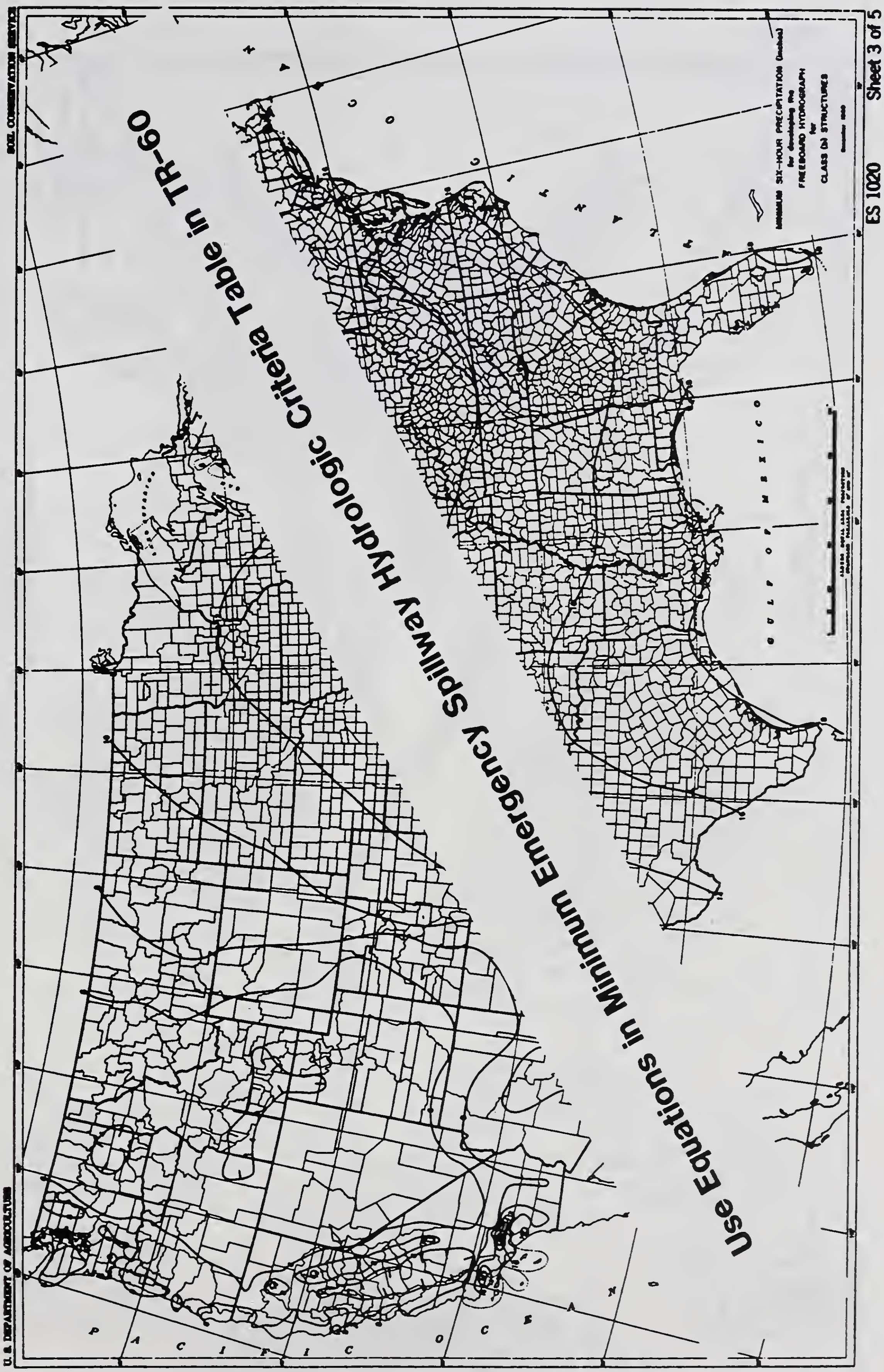


FIGURE 21.5 (3 of 5)

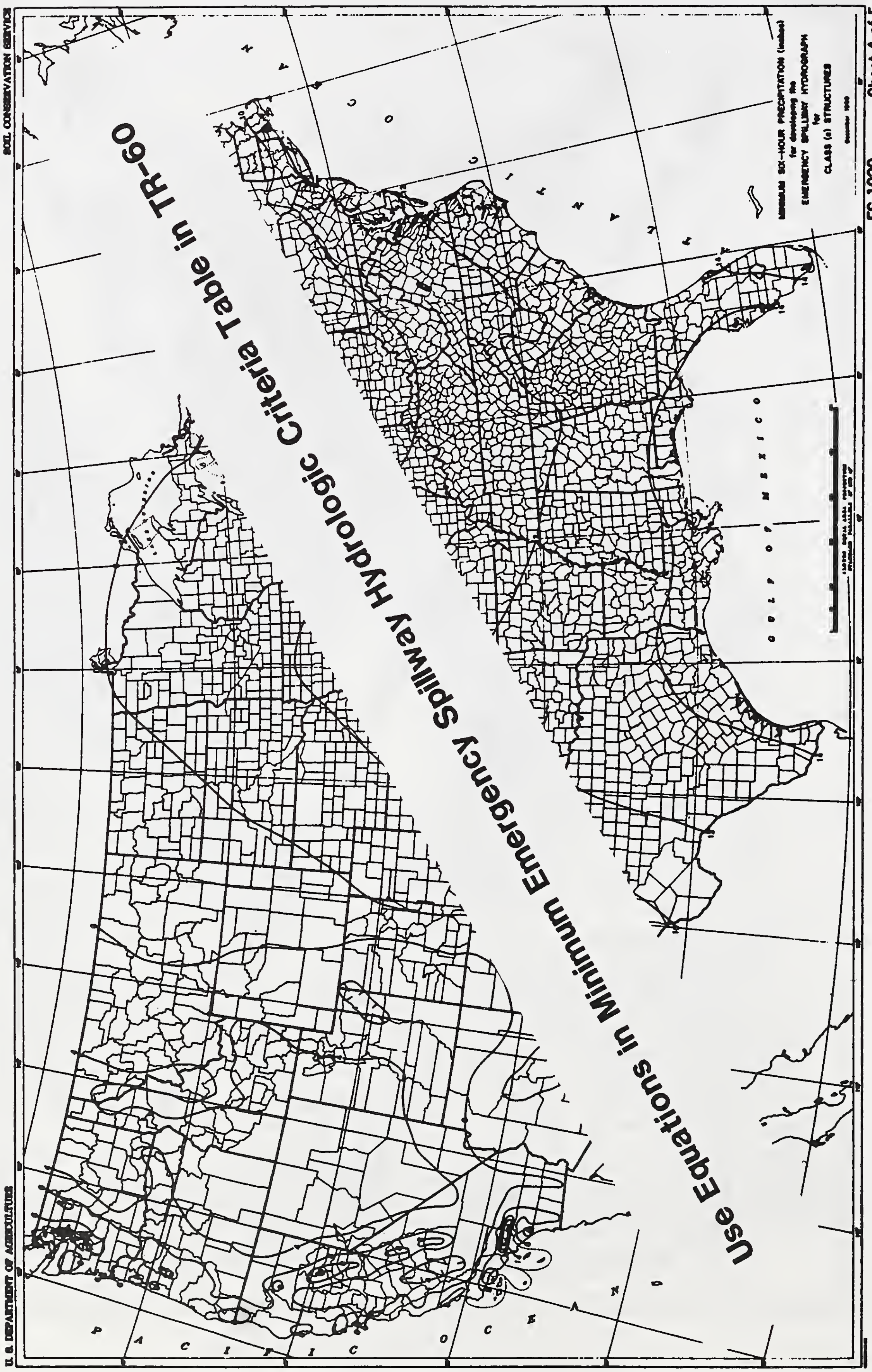


FIGURE 21.5 (4 of 5)

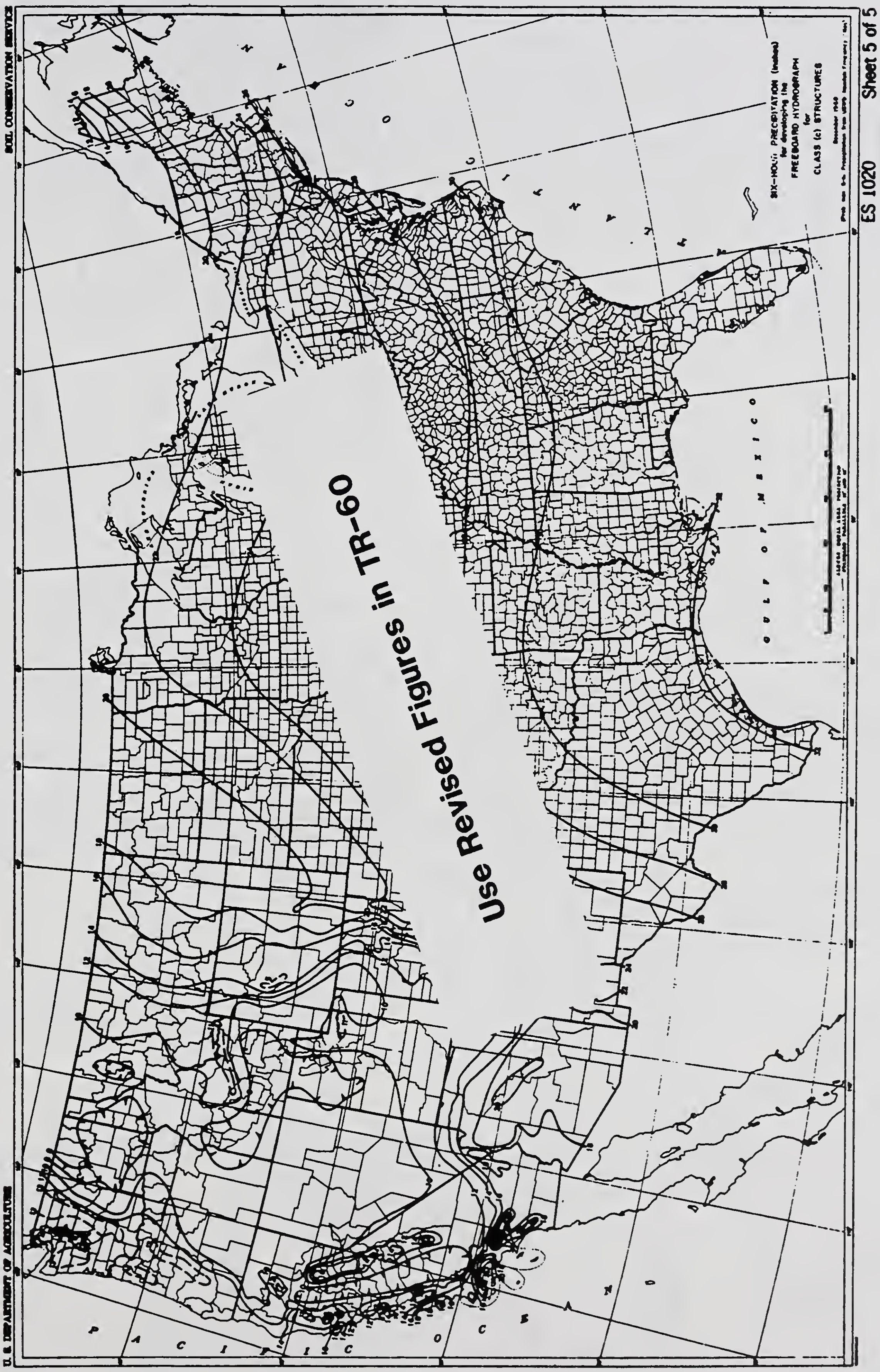


FIGURE 21.5 (5 of 5)

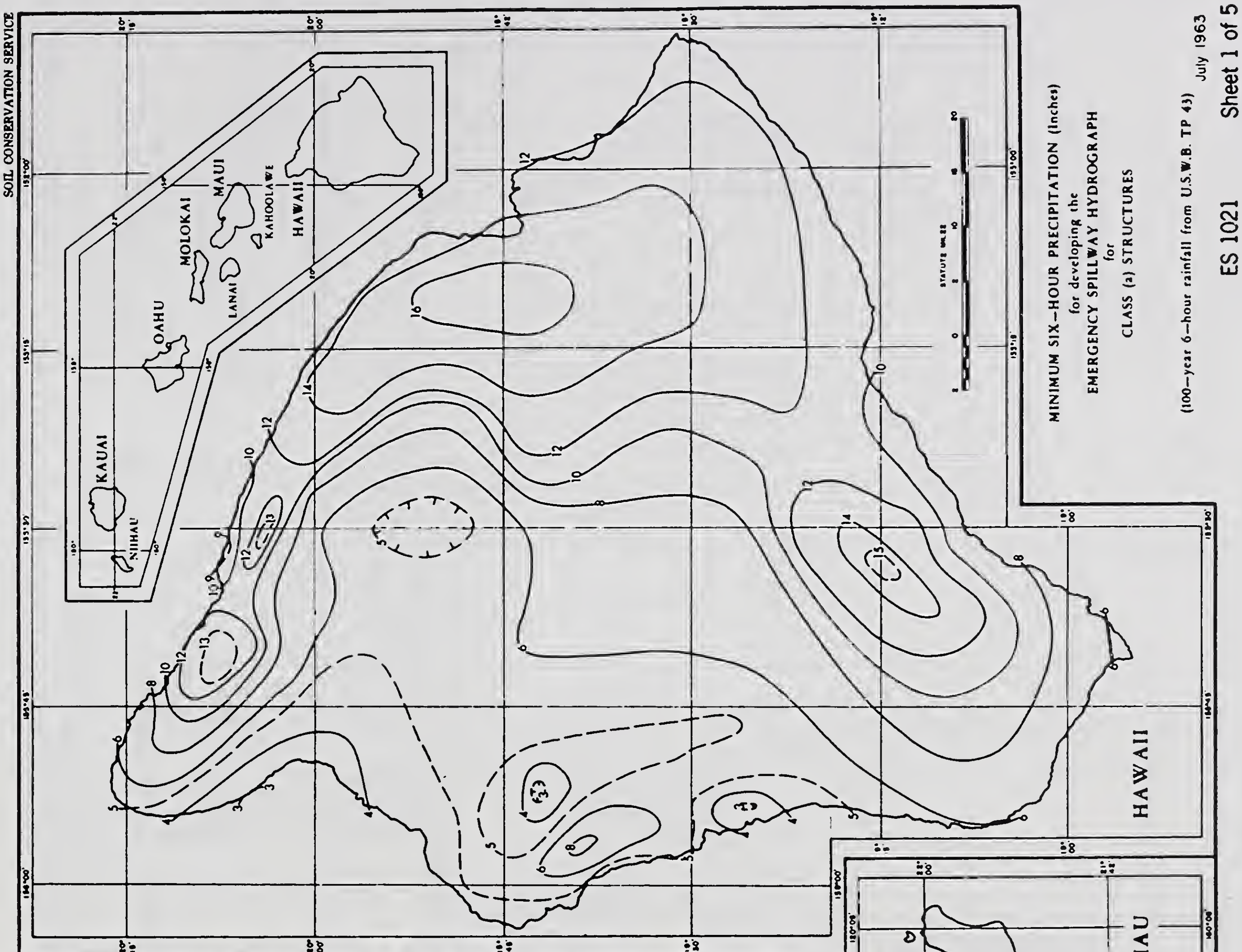


FIGURE 21.6 (1 of 5)

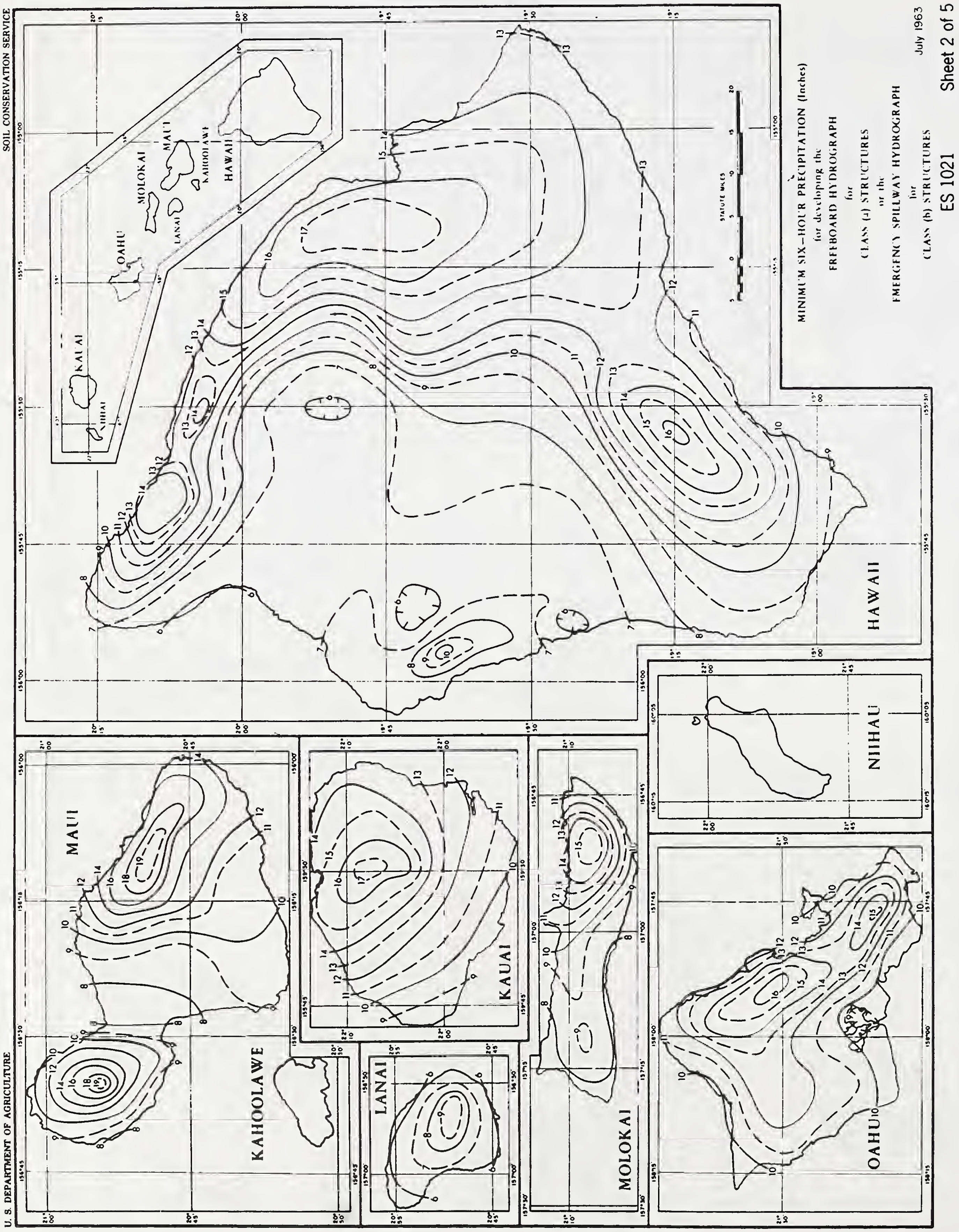


FIGURE 21.6 (2 of 5)

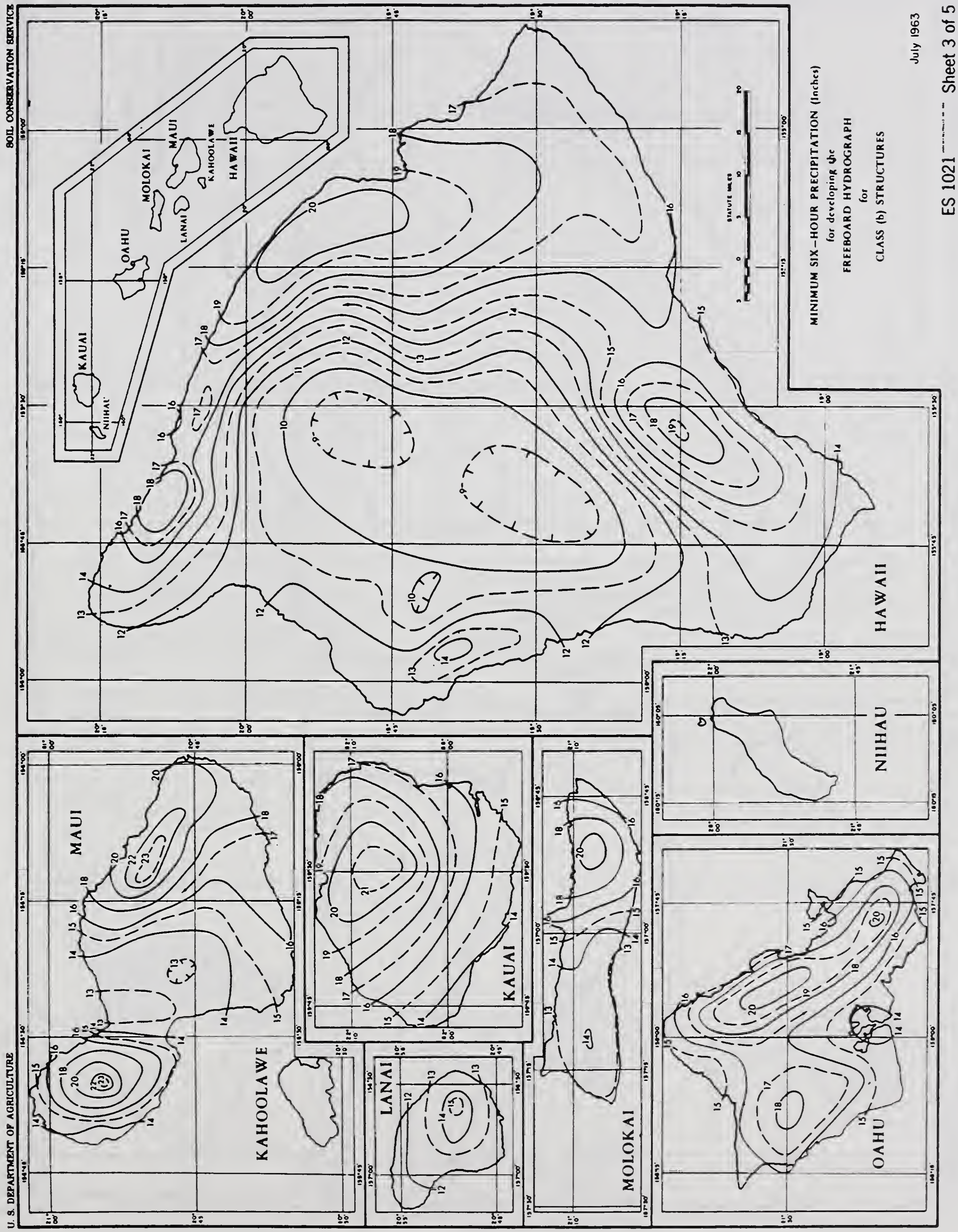


FIGURE 21.6 (3 of 5)

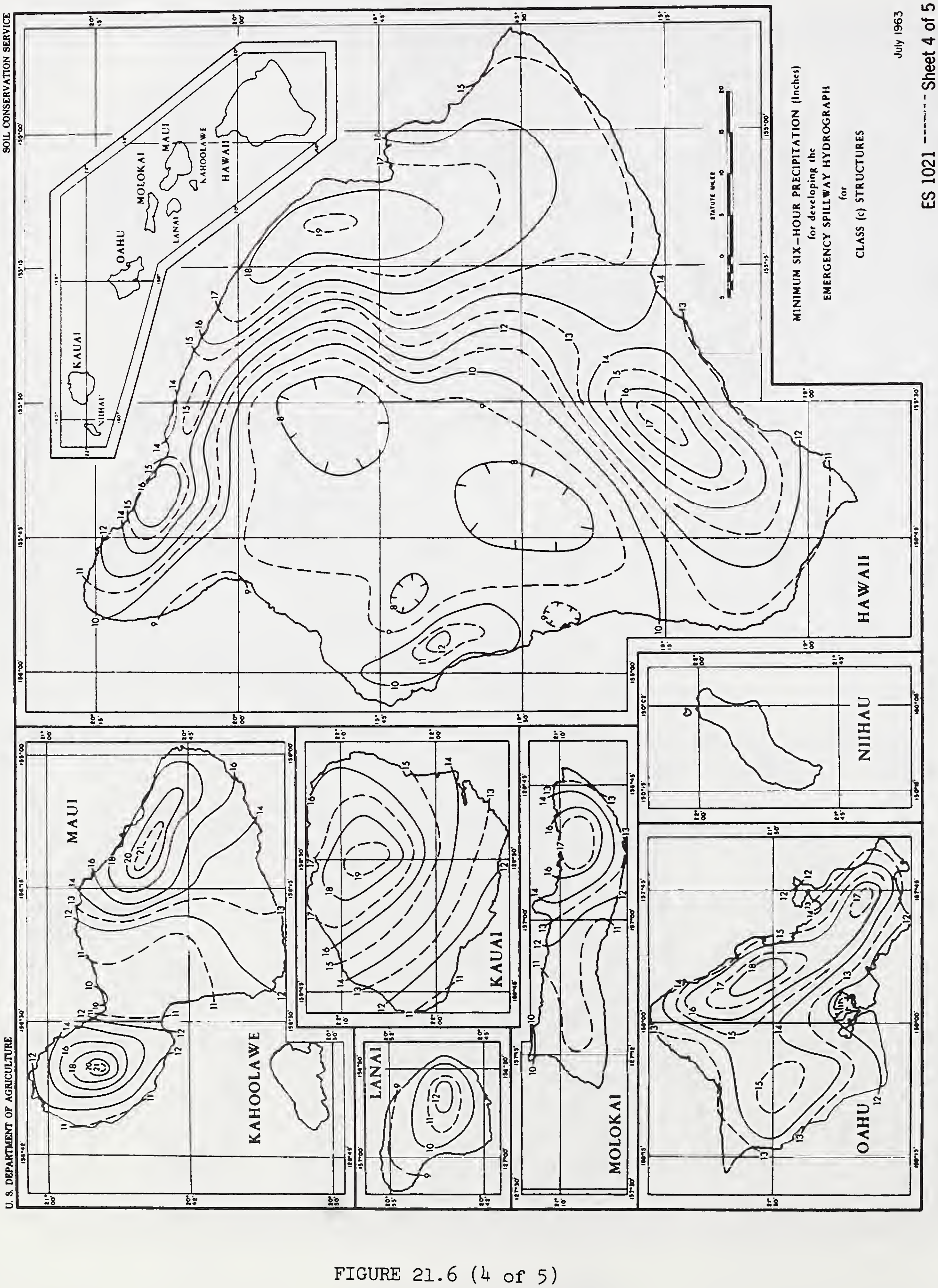


FIGURE 21.6 (4 of 5)

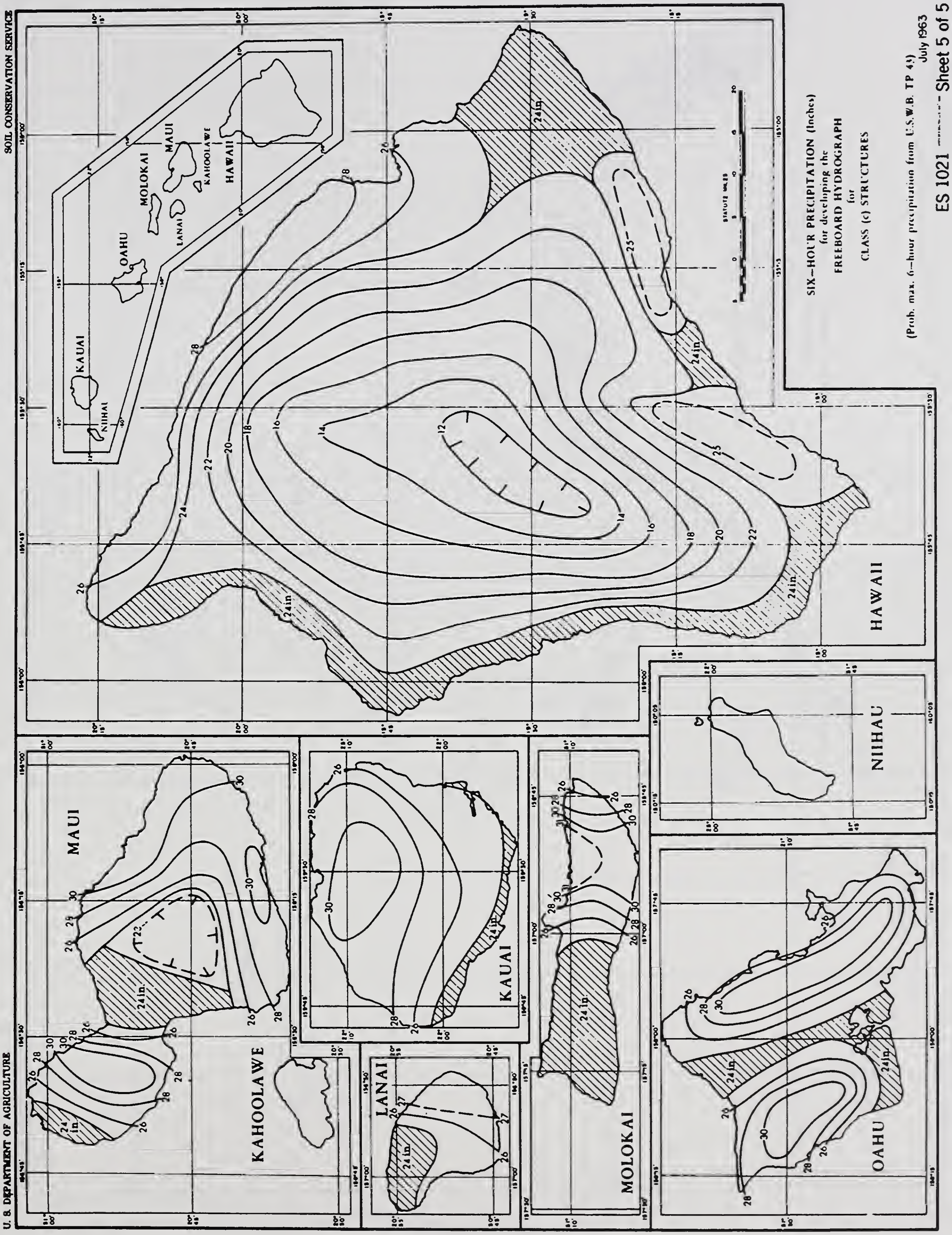


FIGURE 21.6 (5 of 5)

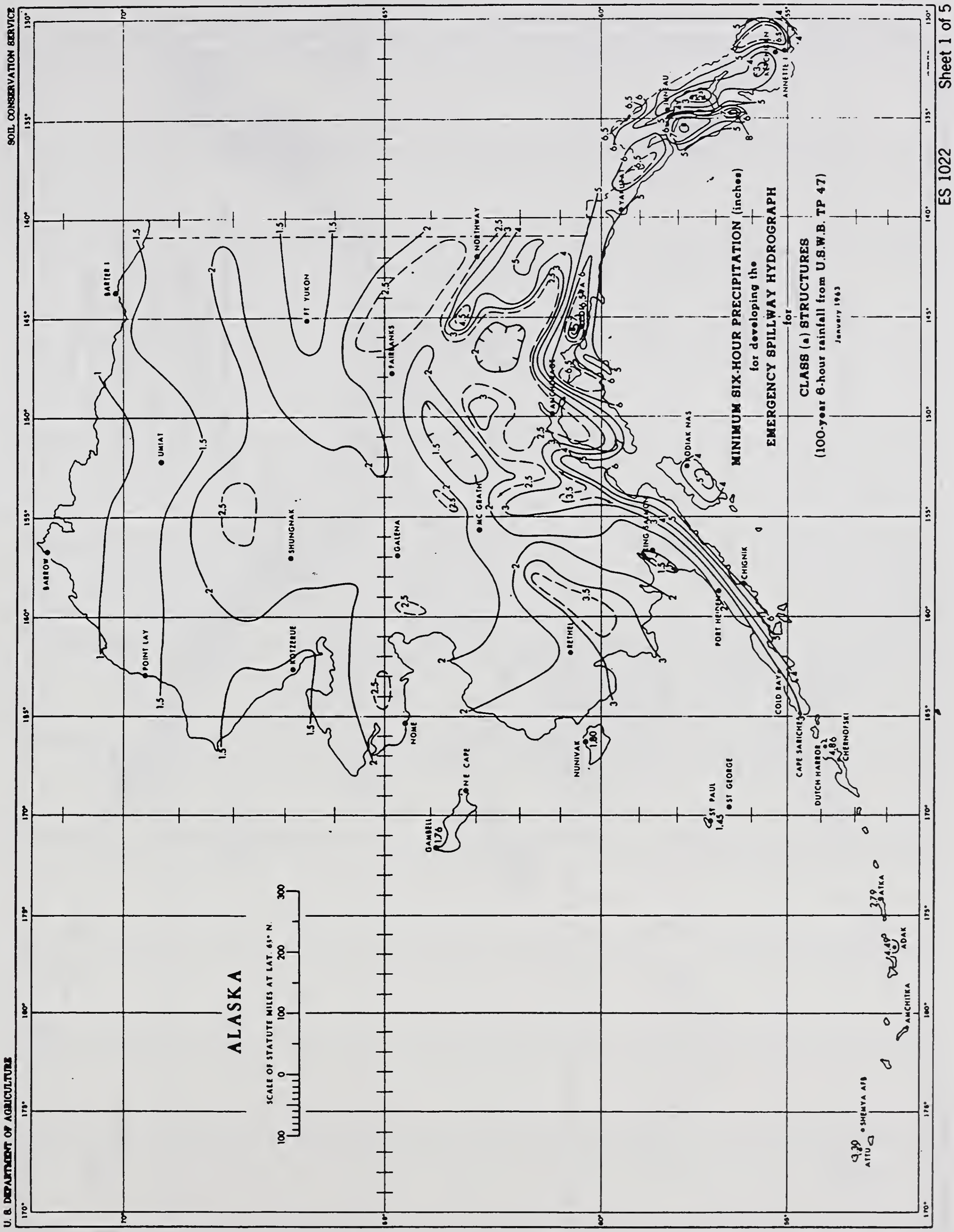


FIGURE 21.7 (1 of 5)

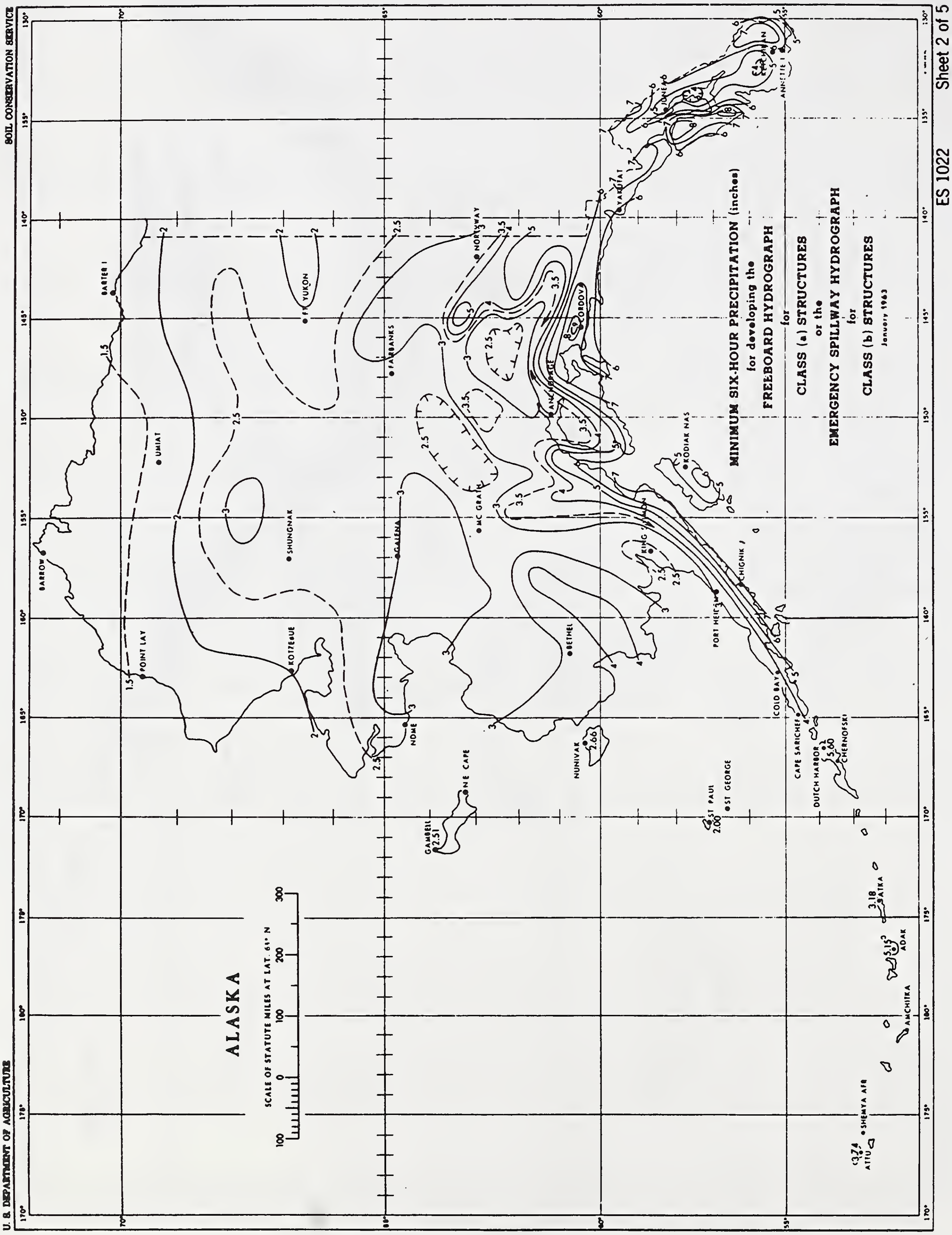


FIGURE 21.7 (2 of 5)

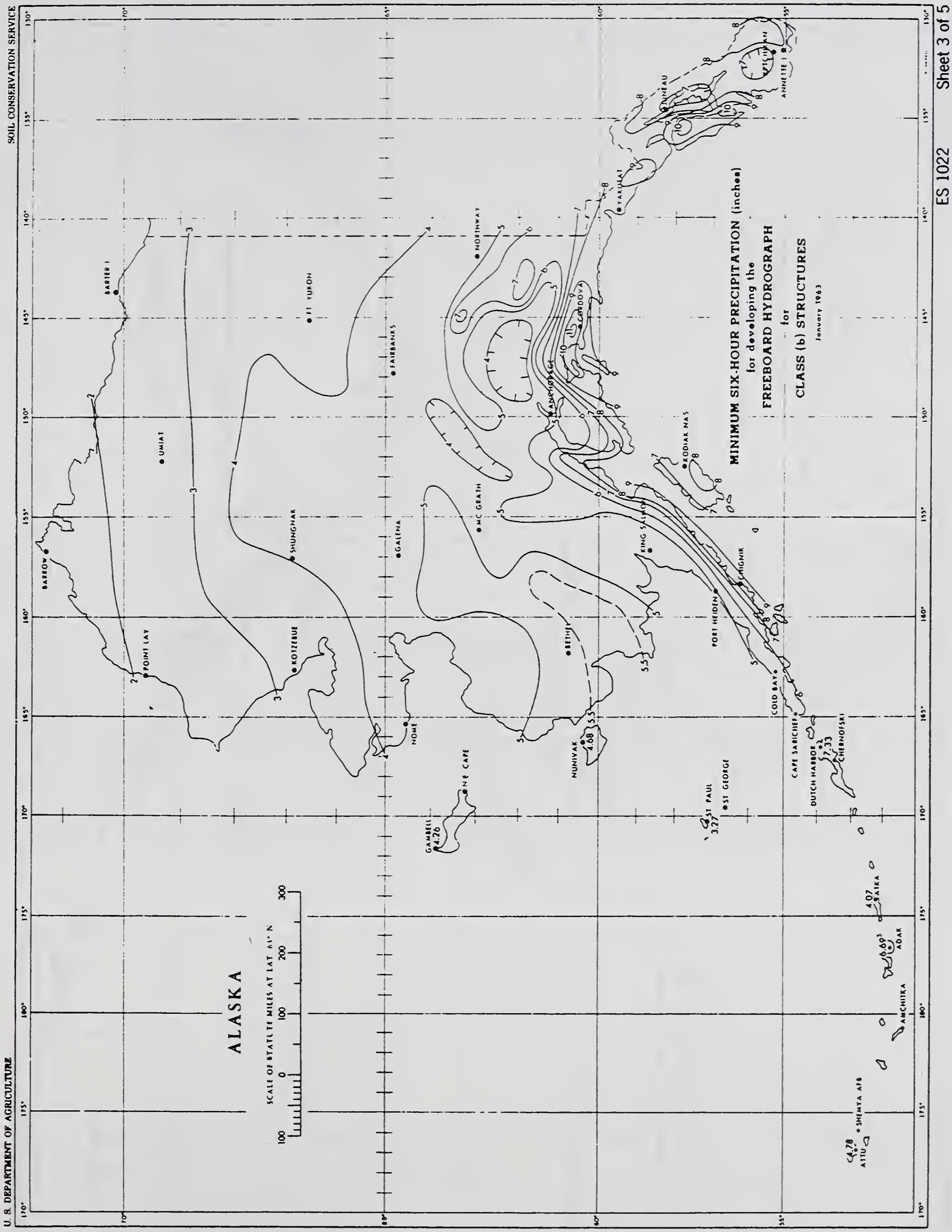


FIGURE 21.7 (3 of 5)

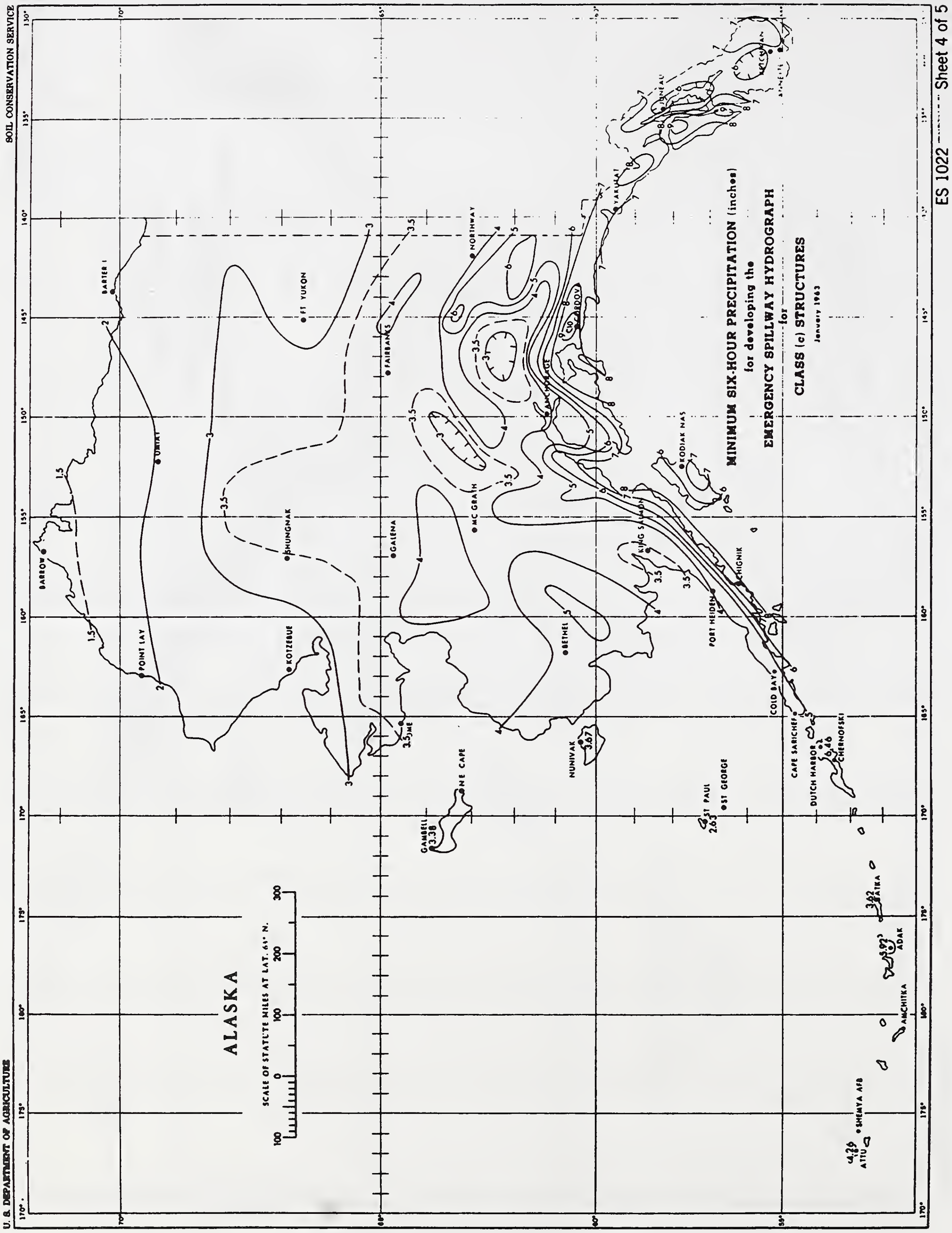


FIGURE 21.7 (4 of 5)

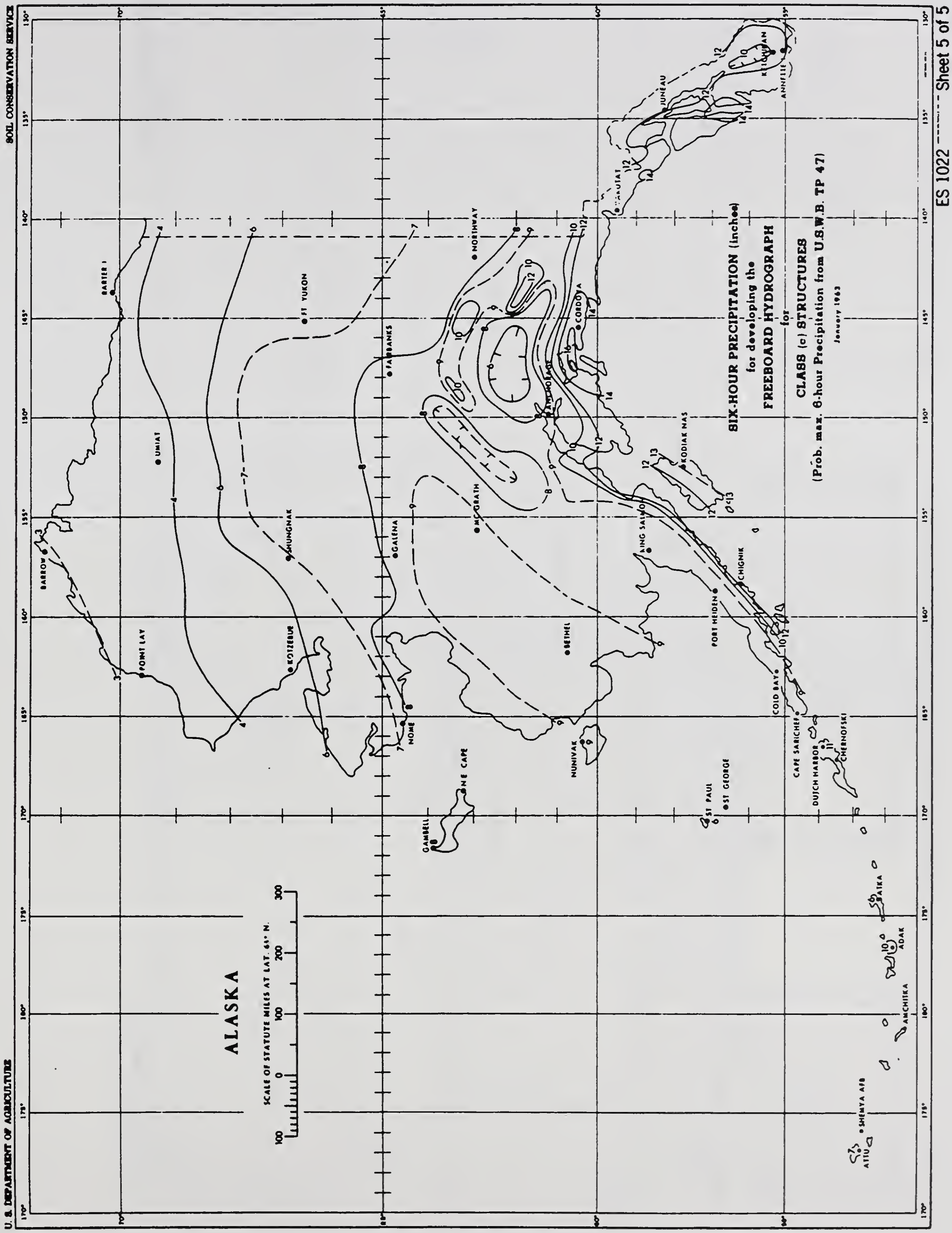


FIGURE 21.7 (5 of 5)

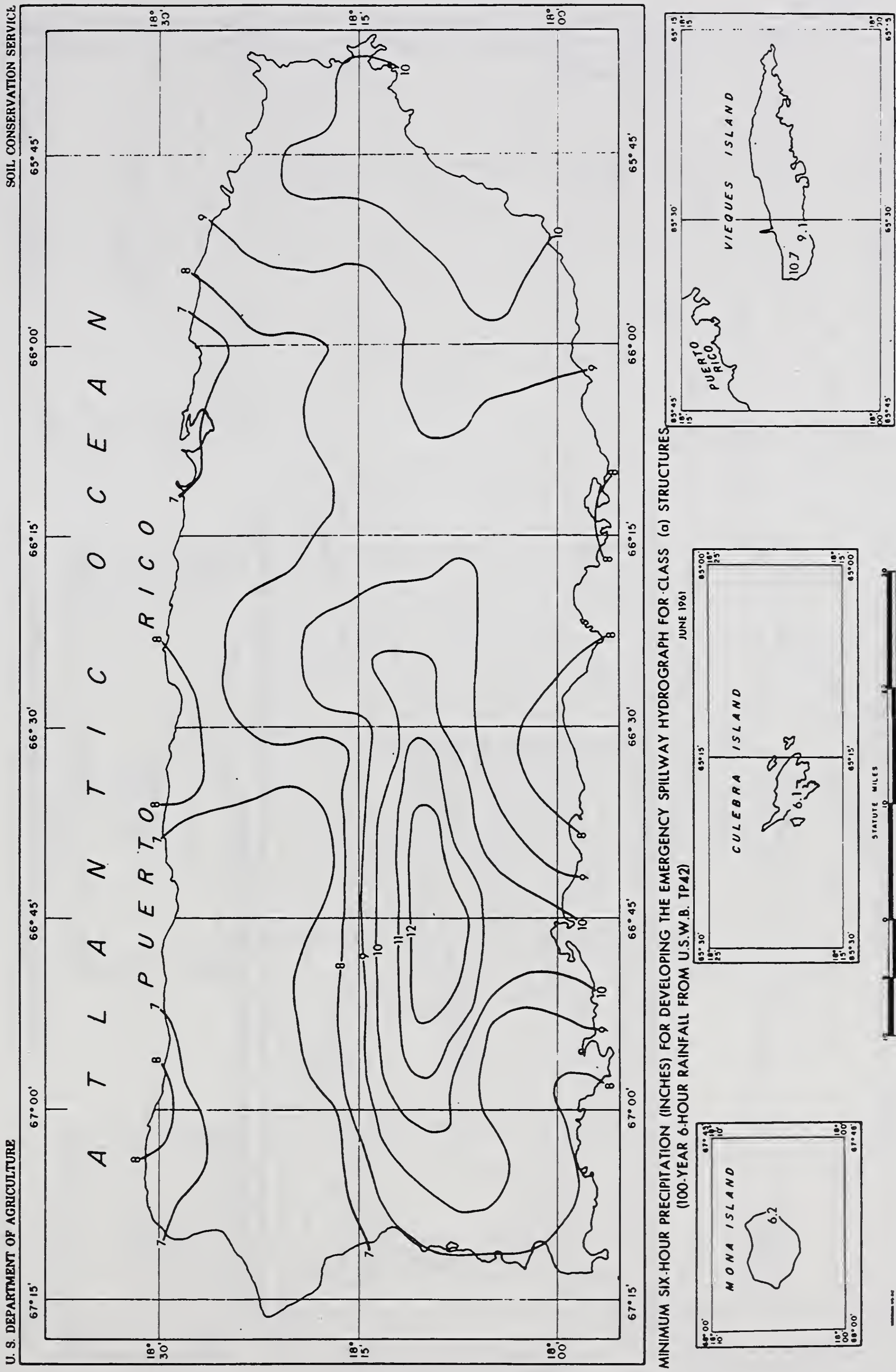


FIGURE 21.8 (1 of 5)

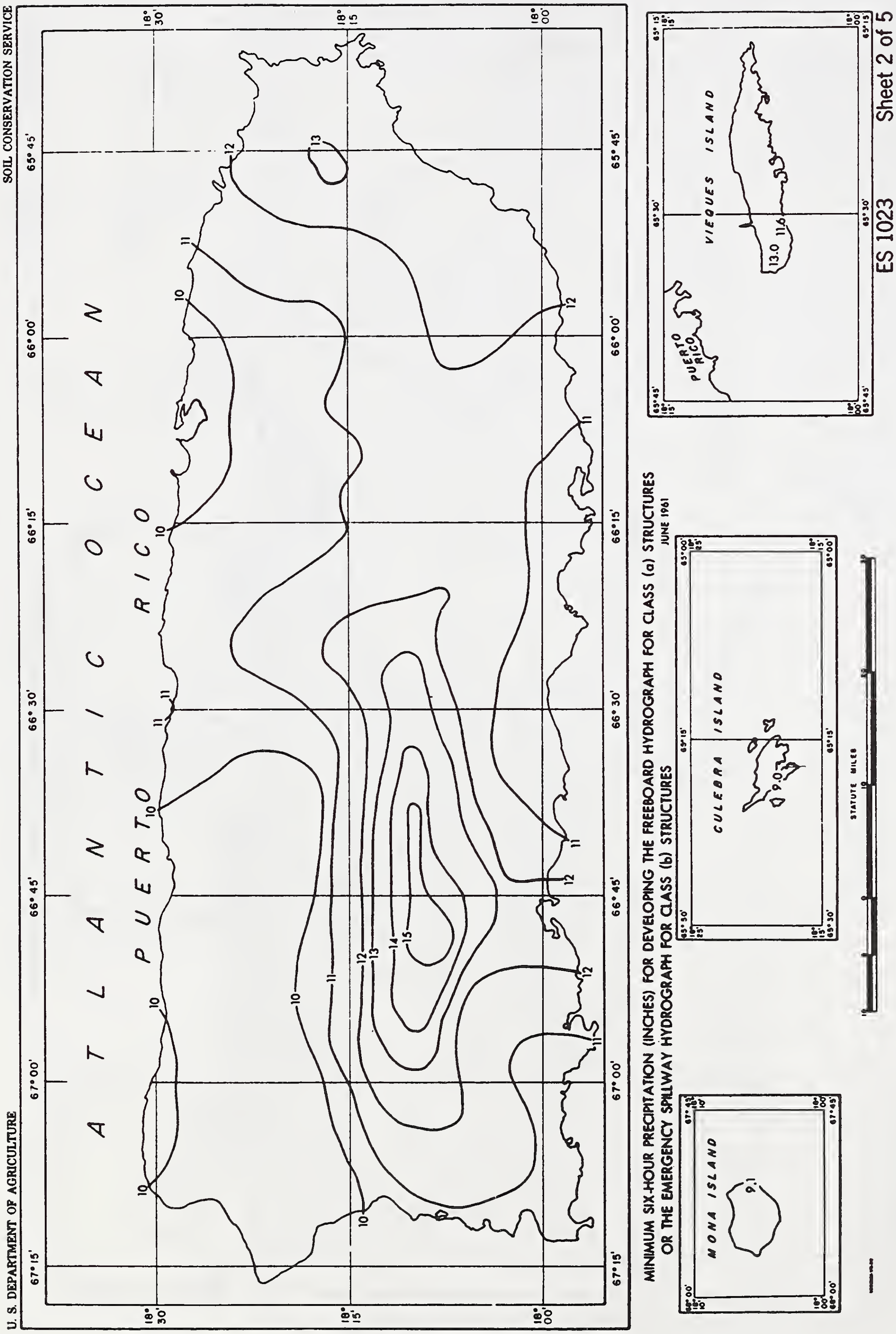


FIGURE 21.8 (2 of 5)

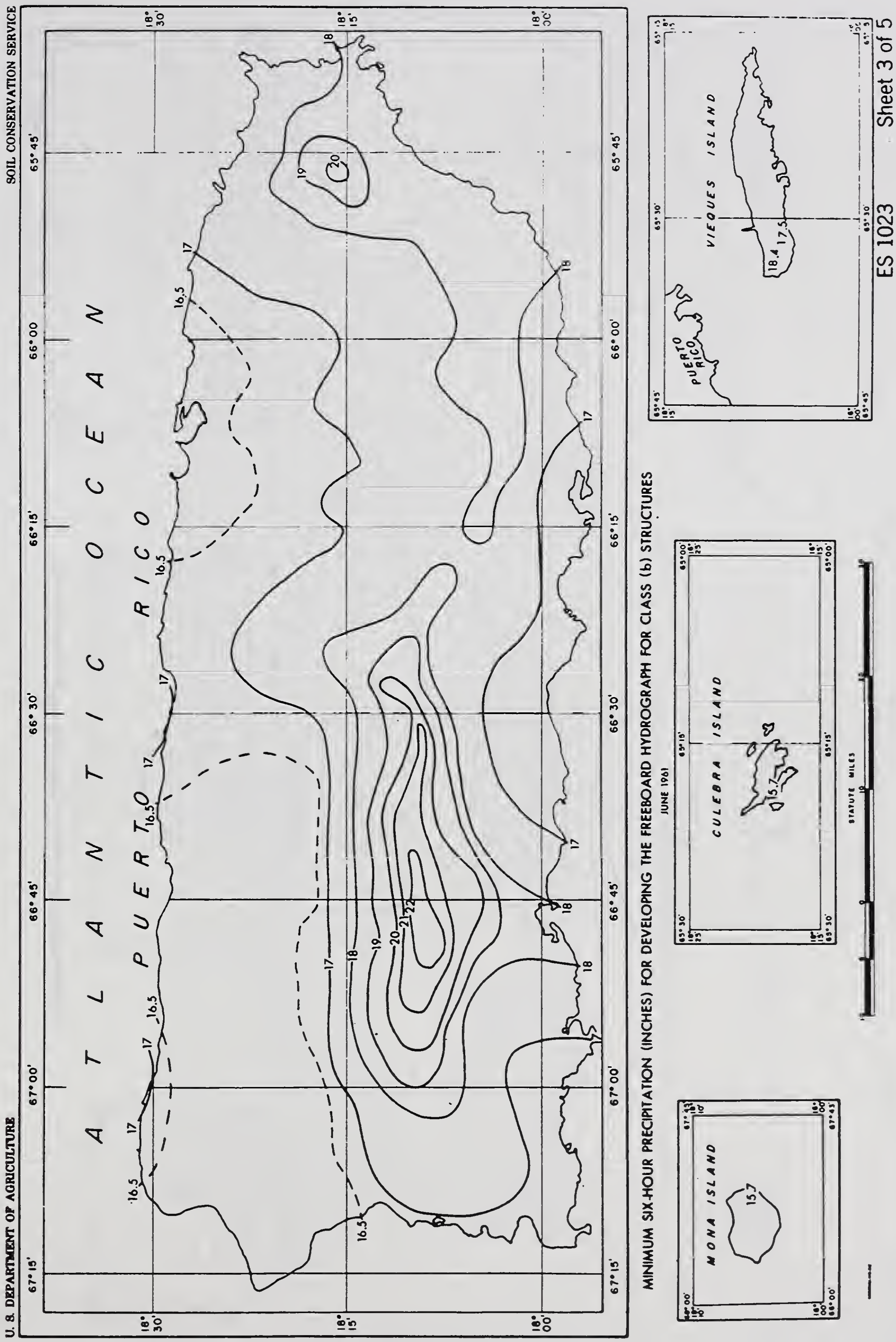


FIGURE 21.8 (3 of 5)

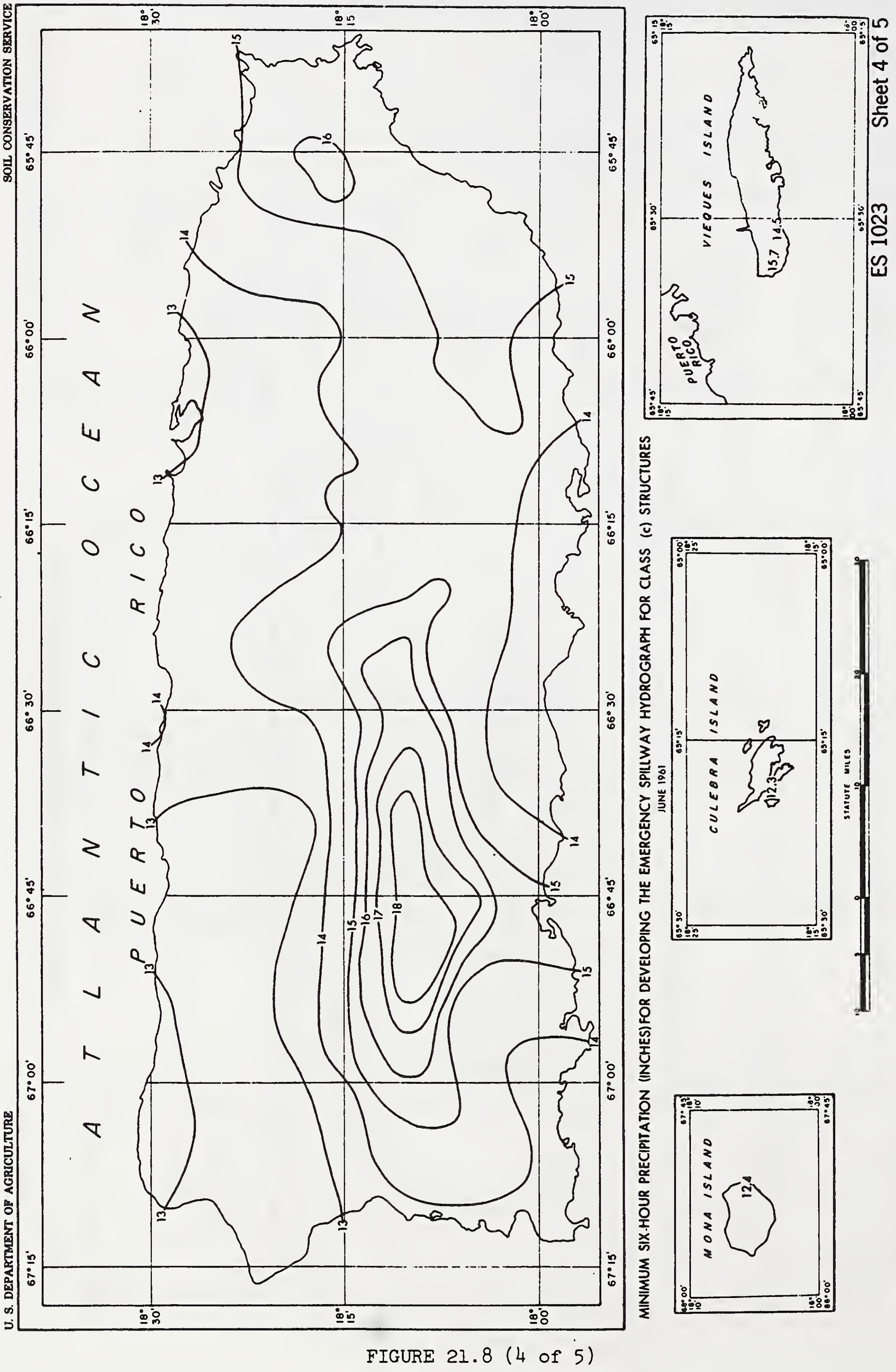


FIGURE 21.8 (4 of 5)

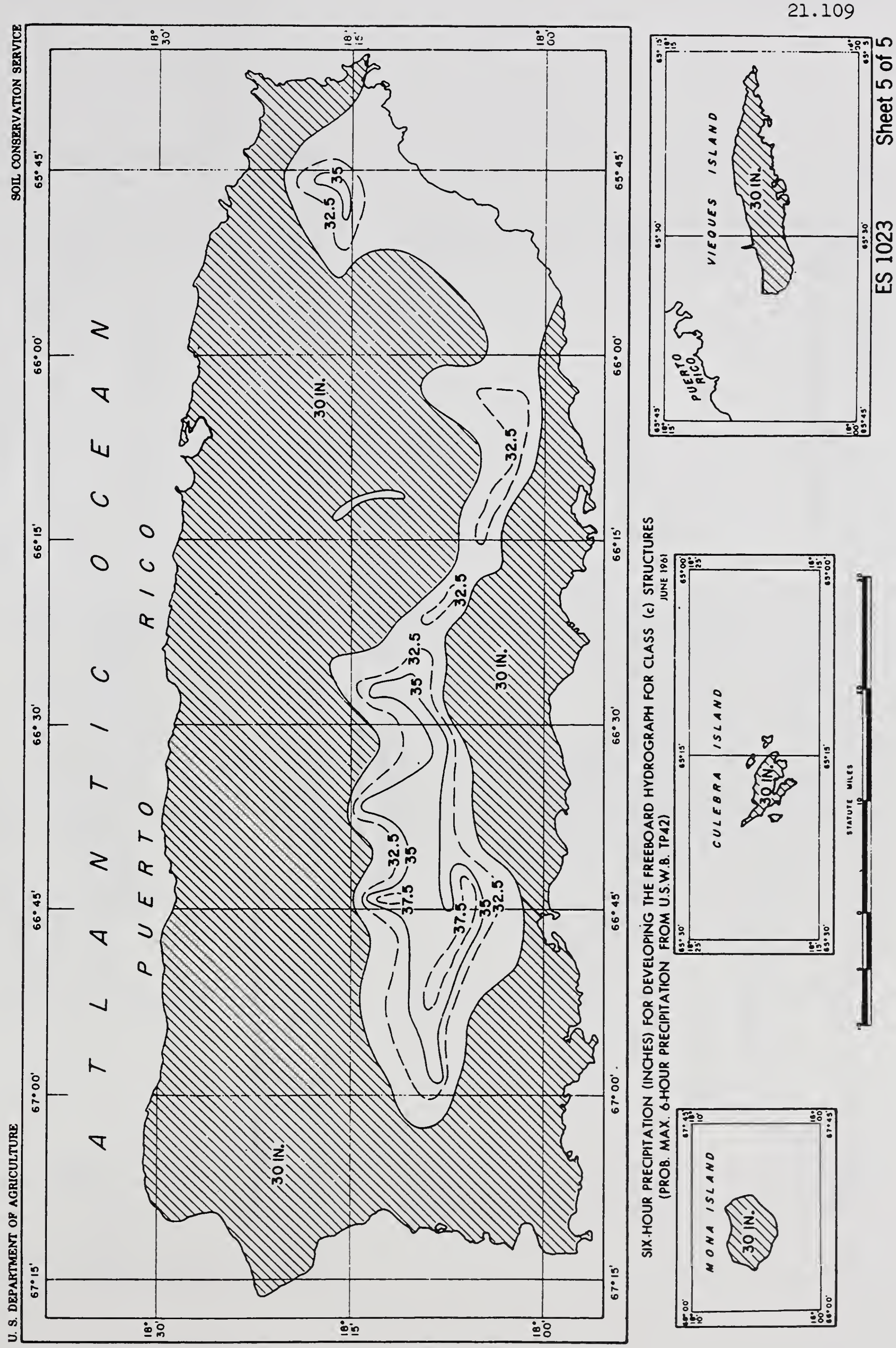


FIGURE 21.8 (5 of 5)

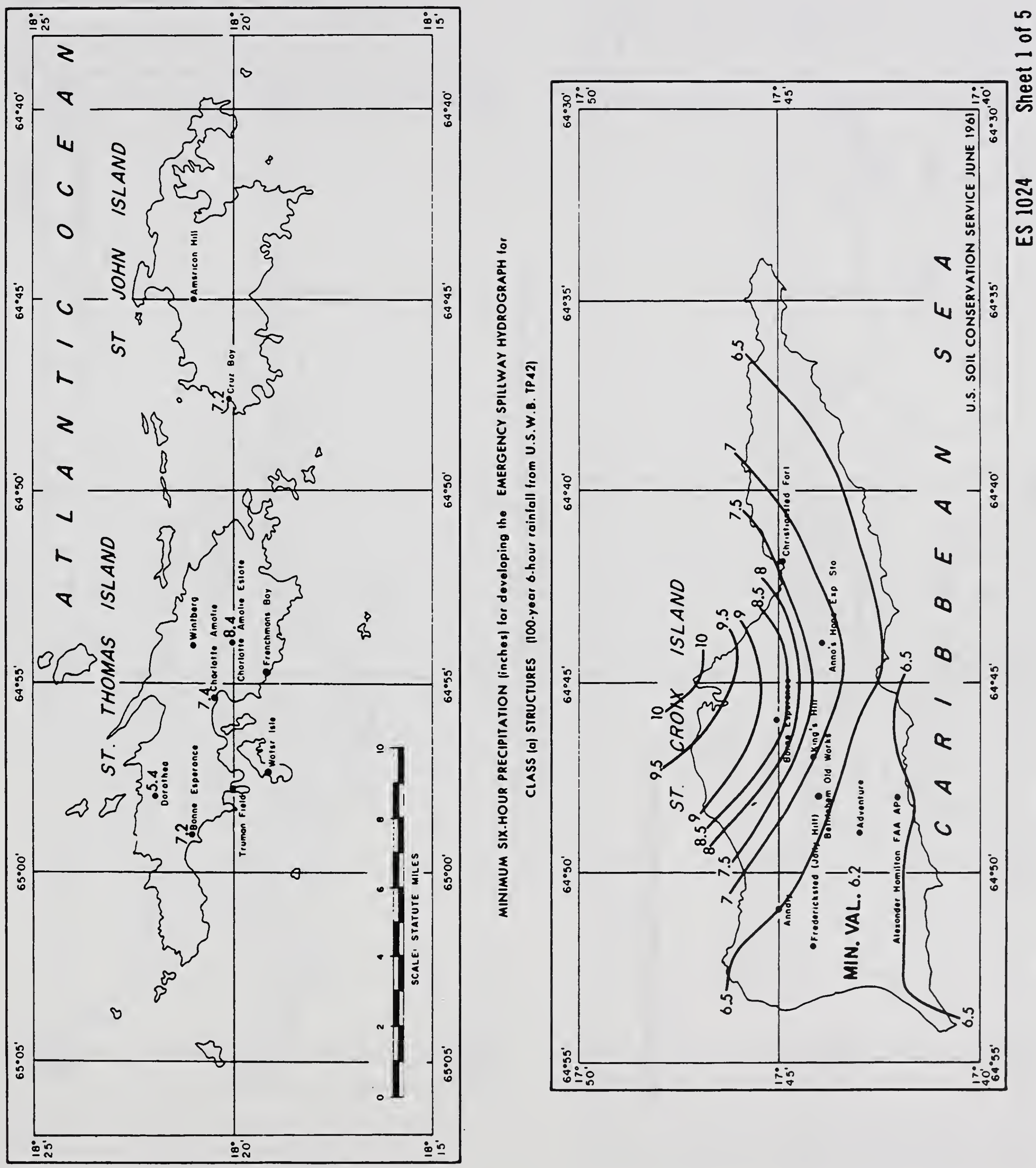


FIGURE 21.9 (1 of 5)

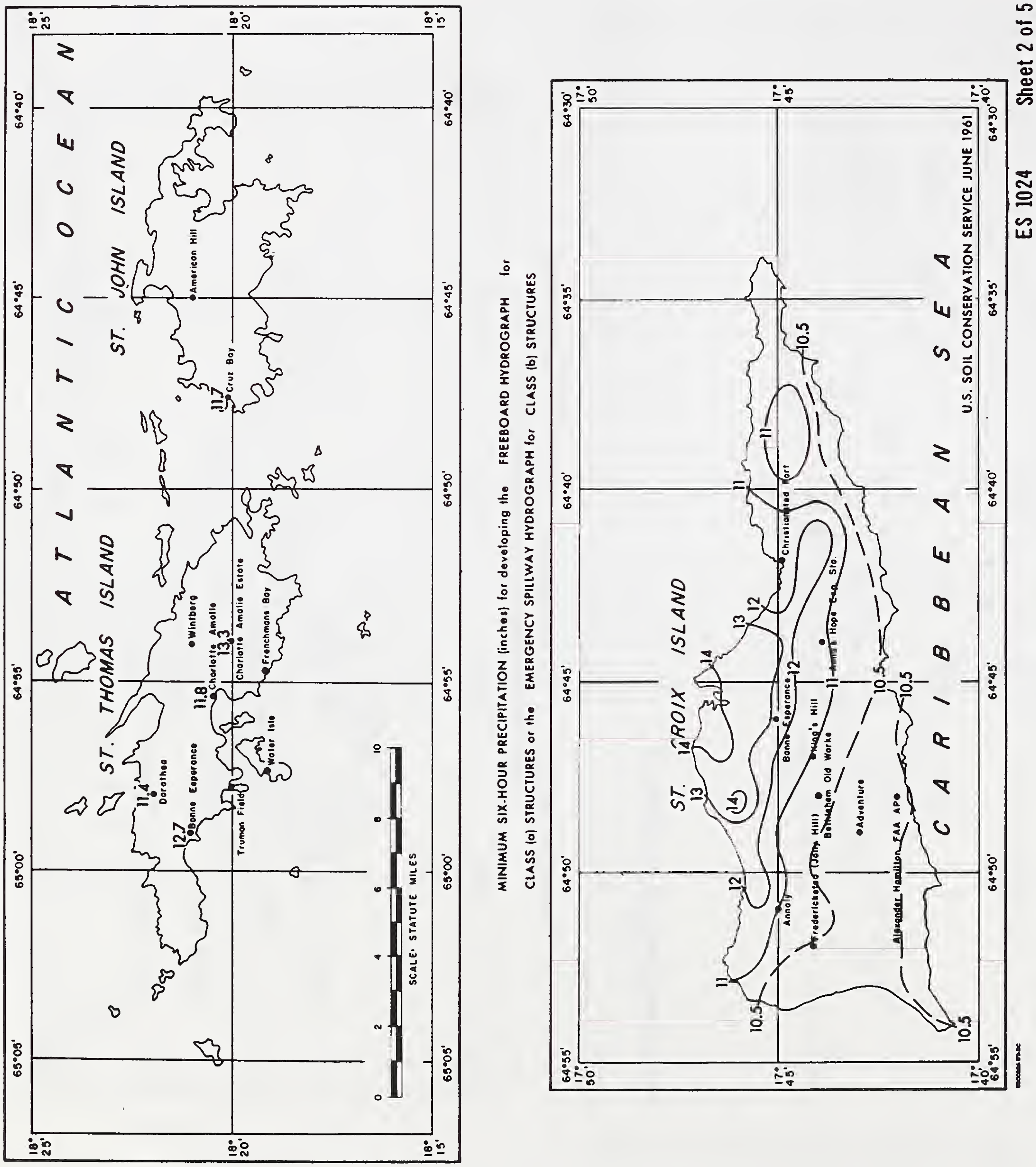


FIGURE 21.9 (2 of 5)

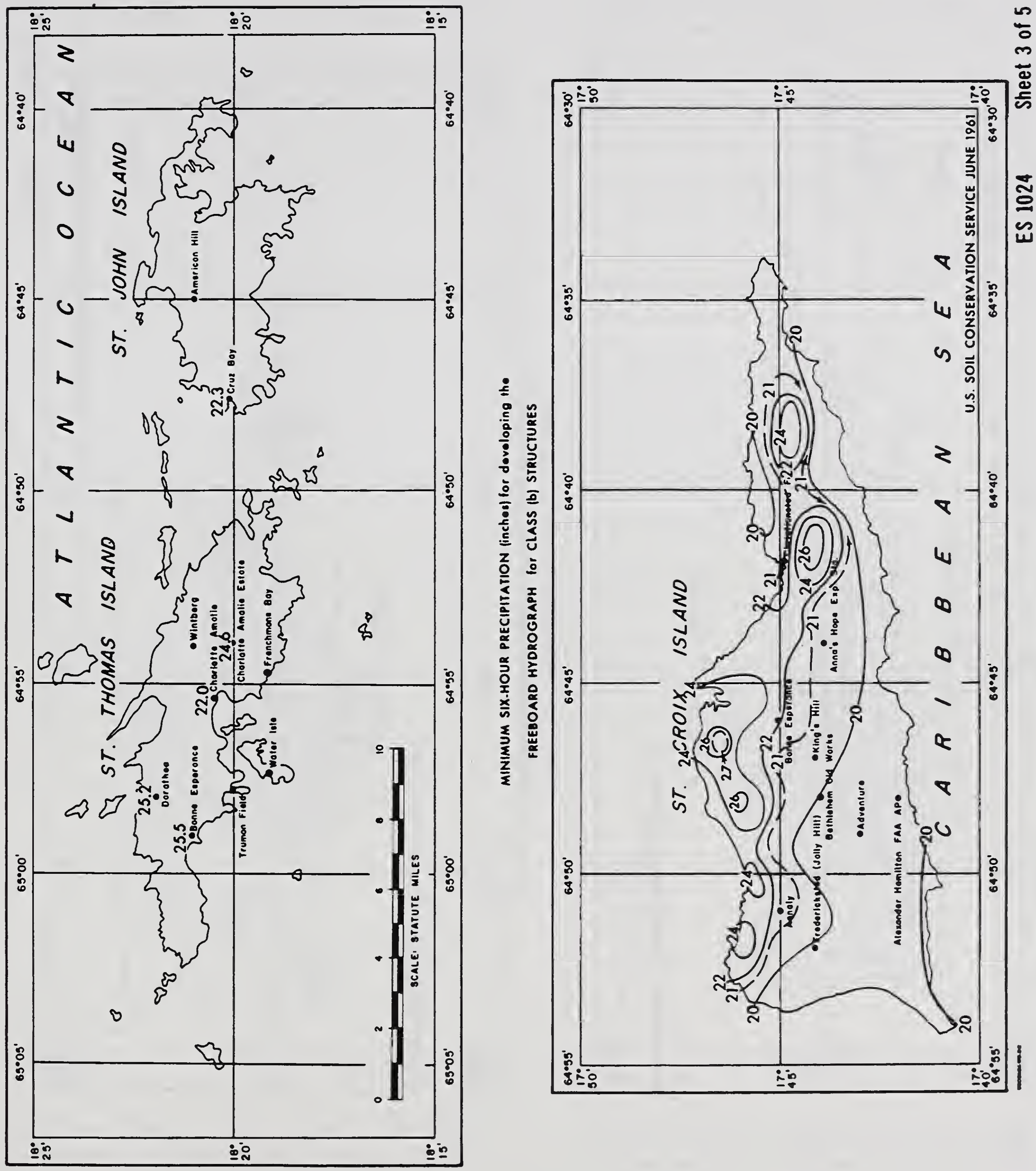


FIGURE 21.9 (3 of 5)

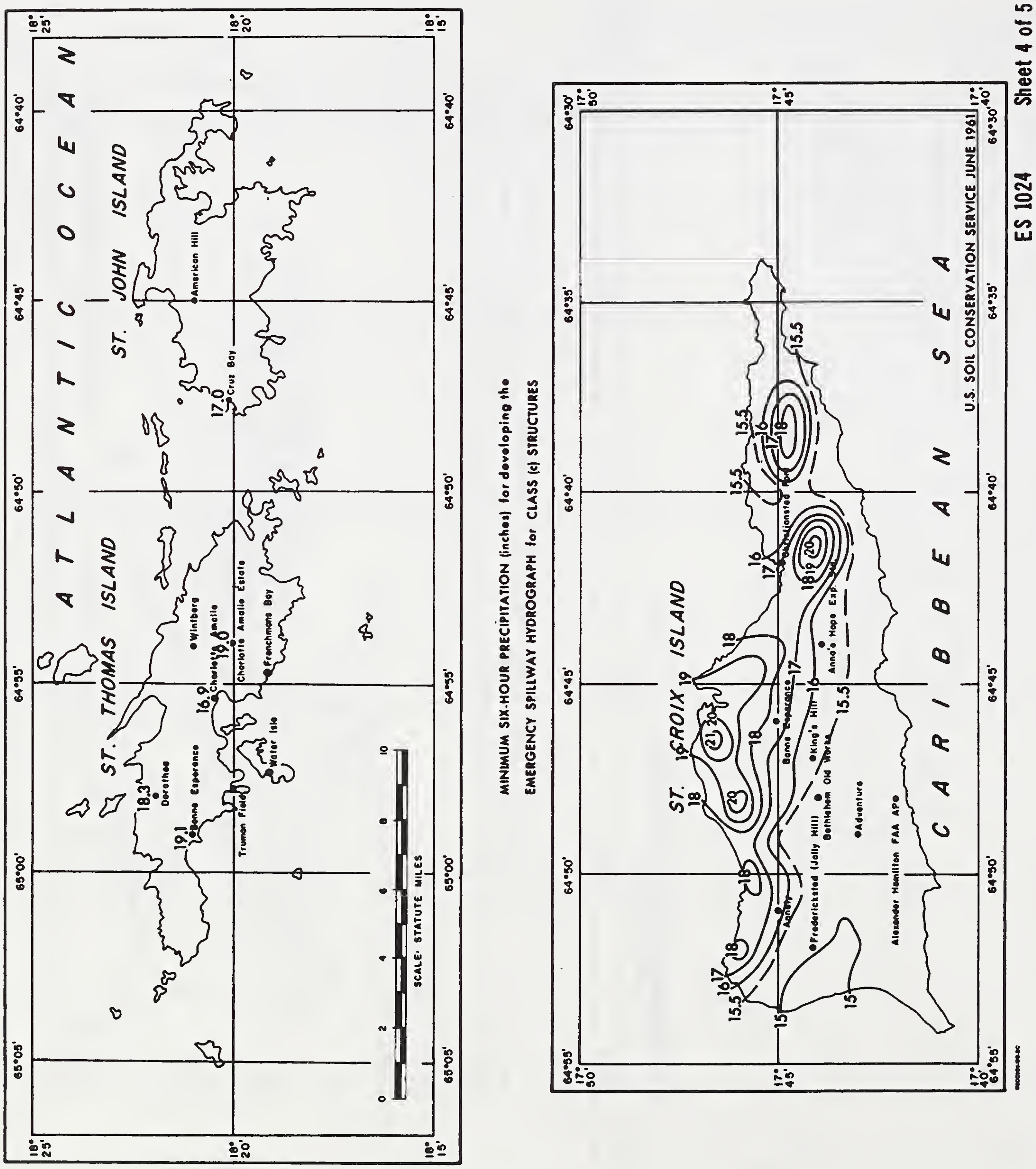


FIGURE 21.9 (4 of 5)

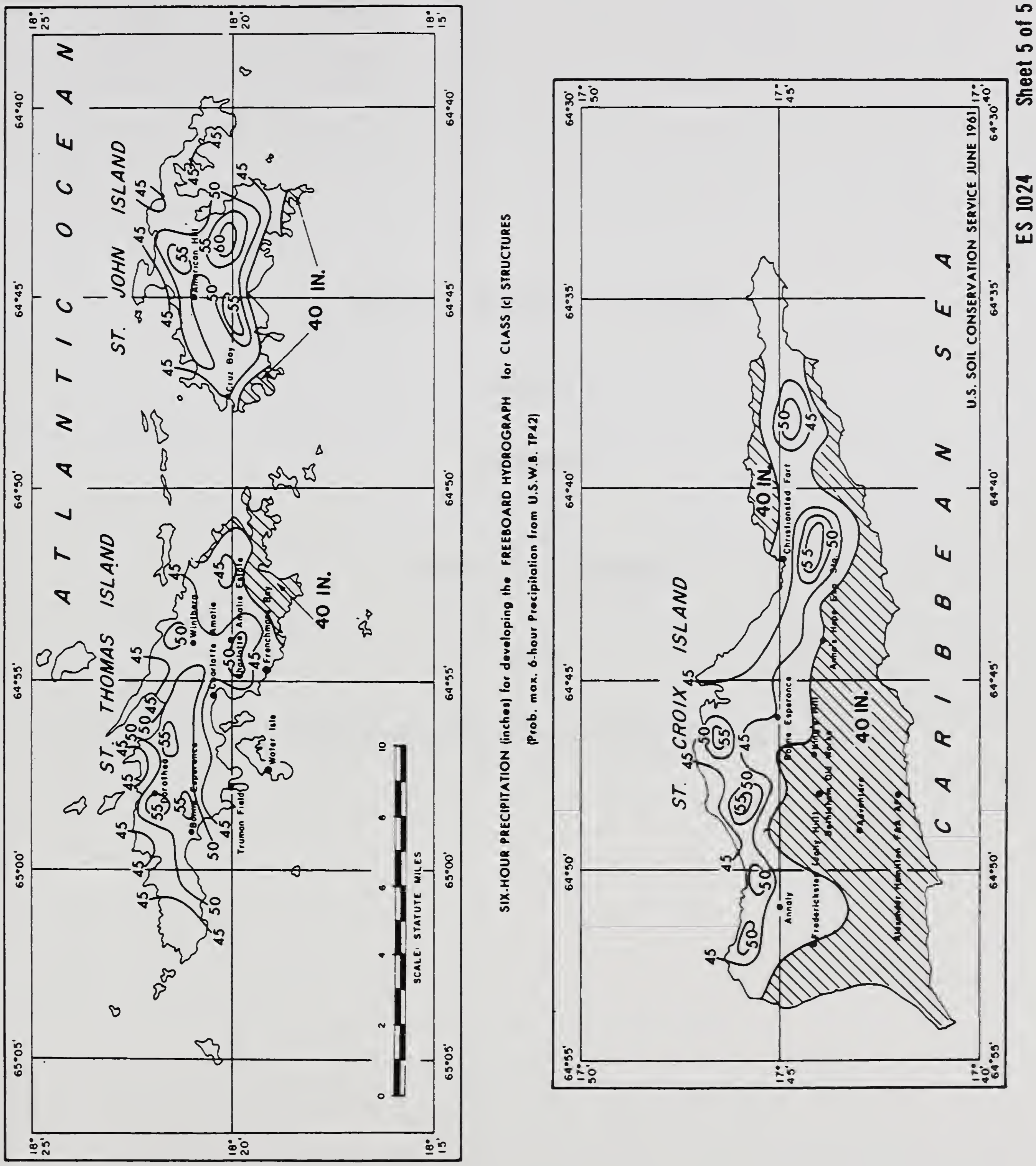


FIGURE 21.9 (5 of 5)

NATIONAL ENGINEERING HANDBOOK

SECTION 4

HYDROLOGY

CHAPTER 22. GLOSSARY

1956

Reprinted with minor revisions, 1971

NATIONAL ENGINEERING HANDBOOK

SECTION 4

HYDROLOGY

CHAPTER 22. GLOSSARY

A selected list of definitions of words and terms used in hydrologic evaluations of watershed projects is given. Other useful definitions are given in:

> National Handbook of Conservation Practices, Information Division, Soil Conservation Service, U. S. Department of Agriculture, Washington, D. C. 20250.
>
> Soil and Water Conservation Glossary, Soil Conservation Society of America, 7515 Northeast Ankeny Road, Ankeny, Iowa 50021.
>
> Nomenclature for Hydraulics (1962), ASCE Manual No. 43 ($6.00), American Society of Civil Engineers, United Engineering Center, 345 East 47th Street, New York, New York 10017.

Underlined words and terms in a definition are defined elsewhere in the list.

acre-foot -- The amount of water that will cover 1 acre to a depth of 1 foot. Equals 43,560 cubic feet. Abbreviated AF.

AF -- Abbreviation for acre-foot or acre-feet.

annual flood -- The highest peak discharge in a water year.

annual runoff -- The total natural discharge of a stream for a year, usually expressed in inches depth or AF. See water yield.

annual series -- A frequency series in which only the largest value in each year is used, such as the annual floods.

annual yield -- The total amount of water obtained in a year from a stream, spring, artesian well, etc. Usually expressed in inches depth, AF, millions of gallons, or cubic feet.

antecedent moisture condition (AMC) -- The degree of wetness of a watershed at the beginning of a storm. (See Chapter 4, Storm rainfall data).

area rainfall -- The average rainfall over an area, usually as derived from, or discussed in contrast with, point rainfall.

base flow -- Stream discharge derived from groundwater sources. Sometimes considered to include flows from regulated lakes or reservoirs. Fluctuates much less than storm runoff.

cfs -- Abbreviation for cubic feet per second. A unit of water flow. Sometimes called "second-feet."

cfs day -- Often called a second-foot-day. The volume of water represented by a flow of 1 cubic foot per second for a period of one day.

consumptive use -- A term used mainly by irrigation engineers to mean the amount of water used in crop growth plus evaporation from the soil. See evapotranspiration.

cover -- The vegetation, or vegetational debris such as mulch, that exists on the soil surface. In some classification schemes, such as table 9-1, fallow or bare soil is taken as the minimum cover class.

cross section (stream or valley) -- The shape of a channel, stream, or valley, viewed across the axis. In watershed investigations it is determined by a line approximately perpendicular to the main path of water flow, along which measurements of distance and elevation are taken to define the cross-sectional area.

damage reach -- A length of floodplain or valley selected for damage evaluation. (See Chapter 6, Stream reaches and cross sections).

degree-day -- As used in snowmelt studies, a day with an average temperature one degree above 32° F. The average is usually obtained by averaging the maximum and minimum for the day. A day with an average of 40° F. gives 8 degree-days.

depth-area curve -- A graph showing the change in average rainfall depth as size of area changes.

design storm -- A given rainfall amount, areal distribution, and time distribution, used to estimate runoff. The rainfall amount is either a given frequency (25-, 50-year, etc.) or a special large value. (See Chapter 21, Design hydrographs).

direct runoff -- The water that enters the stream channels during a storm or soon after, forming a runoff hydrograph. May consist of rainfall on the stream surface, surface runoff, and seepage of infiltrated water (rapid subsurface flow).

double-mass curve -- A graph in which accumulated amounts of item X are plotted versus accumulated amounts of item Y, the amounts for given times being used.

drainage area -- The area draining into a stream at a given point. The area may be of different sizes for surface runoff, subsurface flow, and base flow, but generally the surface runoff area is used as the drainage area. See watershed.

effective duration -- The time in a storm during which the water supply for direct runoff is produced. Also used to mean the duration of excess rainfall.

effective rainfall -- Another term for direct runoff. Usually not the same quantity on upland streams as on downstream rivers because of variability of seepage flows.

emergency spillway -- A rock or vegetated earth waterway around a dam, built with its crest above the normally used principal spillway. Used to assist the principal spillway in conveying extreme amounts of runoff safely past the dam.

ET -- Abbreviation for evapotranspiration.

evaluation series -- A list of floods or storms that produced floods during a representative period, and used in water project evaluation to obtain estimates of flood damages.

evapotranspiration -- Plant transpiration plus evaporation from the soil. Difficult to determine separately, therefore used as a unit for study. See consumptive use.

excessive precipitation -- Standard USWB term for "Rainfall in which the rate of fall is greater than certain adopted limits, chosen with regard to the normal precipitation (excluding snow) of a given place or area." Not the same as excess rainfall.

excess rainfall -- Direct runoff at the place where it originates.

fallow -- Cropland kept free of vegetation during the growing season. May be a normal part of the cropping system for weed control, water conservation, soil conditioning, etc.

f_c -- Symbol for the low, almost uniform, infiltration rate obtained after prolonged wetting of the soil.

flood -- In common usage, an event where a stream overflows its normal banks. In frequency analysis it means an annual flood that may not overflow the banks.

flood routing -- Determining the changes in a flood wave as it moves downstream through a valley or through a reservoir (then sometimes called reservoir routing). Graphic or numerical methods are used.

flood pool -- Floodwater storage in a reservoir. In a floodwater retarding reservoir, the temporary storage between the crests of the principal and emergency spillways.

floodwater retarding structure -- A dam, usually with an earth fill, having a flood pool where incoming flood water is temporarily stored and slowly released downstream through a principal spillway. The reservoir contains a sediment pool and sometimes storage for irrigation or other purposes.

flood wave -- The rise and fall in streamflow during and after a storm.

frequency -- An expression or measure of how often a hydrologic event of given size or magnitude should, on an average, be equaled or exceeded. For example a 50-year frequency flood should be equaled or exceeded in size, on the average, only once in 50 years. In drought or deficiency studies it usually defines how many years will, on the average, be equal to or less than a given size or magnitude.

frequency line -- The line on probability paper that represents a series of events and their frequencies.

frequency series -- A sequence or array of actual events (floods, etc.) suitable for use in frequency analysis; or, a sequence or array of hypothetical events obtained from a frequency analysis.

ground water -- The water in the saturated zone beneath the water table. A source of base flow in streams.

Hazen equation -- $F_a = (2n - 1) / 2y$. Used to obtain plotting positions for plotting flood values on log-normal paper. (See Chapter 18, Frequency methods).

Hazen method -- As considered in the Hydrology Guide, it consists of using the Hazen equation and log-normal paper (or Hazen paper) to obtain frequencies. More generally, it consists also of skewness computations described by Allen Hazen in his book, "Flood Flows," published in 1930 by John Wiley and Sons, Inc., New York, N. Y.

historical series -- A list of all actual storms (or floods) that caused flood damage in a watershed, in a given period of years, with the date of each storm of flood being known.

hydrograph -- A graph showing, for a given point on a stream or for a given point in any drainage system, the discharge, stage, velocity or other property of water with respect to time.

hydrologic soil-cover complex -- A combination of a hydrologic soil group and a type of cover.

hydrologic soil group -- A group of soils having the same runoff potential under similar storm and cover conditions.

hydrology -- The science that deals with the occurrence and behavior of water in the atmosphere, on the ground, and underground. Rainfall intensities, rainfall interception by trees, effects of crop rotations on runoff, floods, droughts, the flow of springs and wells, are some of the topics studied by a hydrologist.

initial abstraction (I_a) -- When considering surface runoff, I_a is all the rainfall before runoff begins. When considering direct runoff, I_a consists of interception, evaporation, and the soil-water storage that must be exhausted before direct runoff may begin. Sometimes called "initial loss," about which see loss.

infiltration -- Rainfall minus interception, evaporation, and surface runoff. The part of rainfall that enters the soil.

interception -- Precipitation retained on plant or plant residue surfaces and finally absorbed, evaporated, or sublimated. That which flows down the plant to the ground is called "stemflow" and not counted as true interception.

irrigation pool -- Reservoir storage used to store water for release as needed in irrigation.

isohyet -- A line on a map, connecting points of equal rainfall amounts.

lag (or lag time) -- Is the time from the centroid of rainfall to the peak of the hydrograph. It can be estimated from time of concentration as 0.6 T_c.

land treatment measure -- A tillage practice, a pattern of tillage or land use, or any land improvement, with a substantial effect of reducing runoff and sediment production or of improving use of drainage and irrigation facilities. Examples are contouring, improved crop rotations, controlled grazing, land leveling, field drainage. In hydrologic computations, nonbeneficial measures (such as straight-row, poor-rotation corn) are included for convenience in evaluation. See table 9-1. In general conservation work "land treatment measure" has a broader meaning that includes measures to improve the soil, control sheet erosion, increase soil fertility.

land use -- A land classification. Cover, such as row crops or pasture, indicates a kind of land use. Roads may also be classified as a separate land use. For a classification scheme, see table 9-1.

log paper -- Short for "full-logarithmic graph paper," which is a graph paper (available commercially) that has logarithmic scales on both horizontal and vertical axes. Sometimes called "log-log paper." The scales may be any number of cycles, but usually in combinations like 1x1, 2x2, 3x3, 3x5, 4x7, etc.

log-normal - Short for "logarithmic-normal probability distribution."

log-normal paper -- Graph paper used in estimating frequencies of floods, etc. Has a logarithmic scale for the flood (or other) amounts, and a cumulative distribution scale (also called frequency or percent chance scale) for the probability plotting positions.

loss -- In hydrology, a loss for one purpose is usually a gain for another, so that the net effect may be more important than the loss. At various times, evapotranspiration, initial abstraction, infiltration, surface storage, direct runoff, seepage, etc. have been called losses according to the aims of a water user. See water loss.

Manning's n -- A coefficient of roughness, used in a formula for estimating the capacity of a channel to convey water. Generally, "n" values are determined by inspection of the channel. See Chapter 14, Stage-discharge relations.

mean daily -- The average or mean discharge of a stream for one day. Usually given in cfs.

NEH-4 -- National Engineering Handbook, Section 4, Hydrology.

NEH-5 -- National Engineering Handbook, Section 5, Hydraulics.

normal -- A mean or average value established from a series of observations, for purposes of comparison of some meteorological or hydrological event.

partial-duration series -- A list of all events, such as floods, occurring above a selected base, without regard to the number, within a given period. In the case of floods, the selected base is usually equal to the smallest annual flood, in order to include at least one flood in each year.

percent chance -- A name often given to the probability scale on <u>log-normal paper</u>. A 2-percent chance flood is a 50-year frequency flood (see <u>frequency</u>) since

$$\frac{100}{\text{percent chance}} = \text{frequency in years}$$

plotting position -- The point computed by an equation and used to locate given data on <u>probability paper</u>. See Chapter 18, Frequency methods.

point rainfall -- Rainfall at a single rain gage.

principal spillway -- A concrete or metal pipe or conduit used with a drop inlet dam or floodwater retarding structure. It conveys, in a safe and nonerosive manner, all ordinary discharges coming into a reservoir and all of an extreme amount that does not pass through the <u>emergency spillway</u>.

probability paper -- Any graph paper prepared especially for plotting magnitudes of events versus their frequencies or probabilities. See <u>log-normal paper</u>.

reach -- A length of stream or valley, selected for convenience in a study. See <u>damage reach</u>, <u>stream reach</u>.

recession curve -- The receding portion of a hydrograph, occurring after <u>excess rainfall</u> has stopped.

recurrence interval -- The average number of years within which a given event will be equaled or exceeded. A 50-year <u>frequency</u> flood has a 50-year recurrence interval; and so on.

regional analysis -- Flood frequency lines for gaged watersheds in a similar area or region are used to develop a flood frequency line for an ungaged watershed in that region. Also used with other types of hydrologic data. Method is a simple (usually graphical and freehand) form of "regression analysis" used by statisticians.

reservoir routing -- Flood routing through a reservoir.

s. d. -- Abbreviation for standard deviation.

second-foot -- See cfs.

second-foot-day -- The volume of water represented by a flow of one cubic foot per second for a period of one day.

sediment pool -- Reservoir storage provided for sediment, thus prolonging the usefulness of floodwater or irrigation pools.

semilog paper -- Short for "semilogarithmic graph paper," which is graph paper having an arithmetic scale along one axis and a logarithmic scale along the other. Either scale is used for the independent variable, as the data require. Commercially available paper has various divisions (5, 6, 7, 10 to the inch) for the arithmetic scale, and various cycles (1, 2, 3, 4, 5) for the logarithmic side.

skew -- When data plot in a curve on log-normal paper, the curvature is skewness. (See Chapter 18, Frequency methods).

small grains -- Wheat, oats, barley, flax, rice, and other close-drilled or broadcast grain crops.

soil-cover complex -- See hydrologic soil-cover complex.

soil-water-storage -- The amount of water the soils (including geologic formations) of a watershed will store at a given time. Amounts vary from watershed to watershed. The amount for a given watershed is continually varying as rainfall or ET takes place.

spillway -- See principal spillway and emergency spillway.

standard deviation -- Statisticians' name for an important measure of dispersion, abbreviated s.d. Data grouped closely about their mean have a small s.d.; grouped less closely, they have a larger s.d. See table 18-3 for calculation of s.d.

standard rain gage -- Also "standard gage." The USWB nonrecording rain gage, having an opening 8 inches in diameter, and a holding capacity of 24 inches of rainfall. The gage is usually examined once daily at a regular time, and the catch (if any) measured by depth in inches and hundredths of an inch.

storage-indication method -- Name often given to a flood-routing method also often called the Puls method (after Louis G. Puls) though it is actually a variation of the method devised by Puls.

stream reach -- A length of stream channel selected for use in hydraulic or other computations.

structural measure -- For flood prevention work, any form of earthwork (dam, ditch, levee, etc.) or installation of concrete, masonry, metal or other material (drop spillway, jetties, riprap, etc.); or installation for forest fire protection (firetowers, roads, firebreaks); or, in some cases, a special planting for nonfarm purposes (stabilization of critical sediment-producing area, etc.).

subsurface runoff -- Water that infiltrates the soil and reappears as seepage or spring flow, and forms part of the flood hydrograph for that storm. Difficult to determine in practice and seldom worked with separately. See direct runoff.

subwatershed -- A watershed that is part of a larger watershed. It is worked on separately when necessary in order to improve computational accuracy for results on a whole watershed basis, or to get results for that area only.

surface runoff -- Total rainfall minus interception, evaporation, infiltration, and surface storage, and which moves across the ground surface to a stream or depression.

surface storage -- Natural or man-made roughness of a land surface, which stores some or all of the surface runoff of a storm. Natural depressions, contour furrows, and terraces are usually considered as producing surface storage, but stock ponds, reservoirs, stream channel storage, etc. are generally excluded.

synthetic series -- A storm or flood series obtained by taking selected values from a frequency line based on historical data.

time of concentration (T_c) -- The time it takes water from the most distant point (hydraulically) to reach a watershed outlet. T_c varies, but often used as constant.

transmission loss -- A reduction in volume of flow in a stream, canal, or other waterway, due to infiltration or seepage into the channel bed and banks. Evaporation is also a transmission loss, but it is ordinarily neglected under the assumption that it is small.

travel time -- The average time for water to flow through a reach or other stream or valley length that is less than the total length. A travel time is part of a T_c but never the whole T_c.

unit hydrograph -- A discharge hydrograph coming from 1 inch of direct runoff distributed uniformly over the watershed, with the direct runoff generated at a uniform rate during the given storm duration. A watershed may have 1-hour, 2-hour, etc. unit hydrographs.

USGS -- United States Department of the Interior, Geological Survey.

USWB -- United States Department of Commerce, Weather Bureau.

water equivalent -- The depth of water, in inches, that results from melting a given depth of snow.

water loss -- Variable meaning, depending on personal interest of water user. Farmers and ranchers usually think of flood runoff as a water loss; many river engineers think of infiltration as a water loss. In Hydrology Guide, the meaning is apparent from the context. See loss.

watershed -- The area contributing direct runoff to a stream. Usually it is assumed that base flow in the stream also comes from the same area. However, the ground-water watershed may be larger or smaller.

watershed measures -- Any vegetative or structural means (including earthwork) of directly improving or conserving the soil and water resources of a watershed. See land treatment measure and structural measure.

water table -- The upper surface of ground water.

water year -- The year taken as beginning October 1. Often used for convenience in streamflow work, since in many areas streamflow is at its lowest at that time. Used by USGS in their WSP.

water yield -- The actual streamflow, at a given place, from a watershed. This is natural annual runoff that may be affected by irrigation uses, reservoir losses, diversions into or out of the watershed, etc.

WSP -- Water-Supply Paper. An annual publication of the USGS, in which streamflow for the water year is given for all gaged streams in a subdivision of the United States or in Hawaii.

CONVERSIONS

THIS:	TIMES THIS:	GIVES YOU THIS:
cfs days	1.983	AF
cfs days	0.03719	inches depth on 1 square mile
cfs days per square mile	0.03719	inches depth
cfs hours	0.08264	AF
cfs hours per square mile	0.001550	inches depth
cfs	1.983	AF per day
cfs	724.0	AF per year (365 days)
cfs	448.8	U. S. gallons per minute
cfs	0.6463	million U. S. gallons per day
csm	0.03719	inches depth per day
csm	13.57	inches depth per year (365 days)
inches per hour	645.3	csm
inches per hour	1.008	cfs per acre
inches depth	53.33	AF per square mile
inches depth on 1 sq. mi.	53.33	AF
AF	0.5042	cfs days
AF	12.10	cfs hours
AF	0.01875	inches depth on 1 square mile
AF	0.3258	million U. S. gallons
AF per day	0.5042	cfs
AF per square mile	0.01875	inches depth
U. S. gallons per minute	0.002228	cfs
million U. S. gallons per day	1.547	cfs
million U. S. gallons per day	3.069	AF
feet per second	0.6818	miles per hour
centimeters	0.3937	inches
hectares	2.471	acres
liters	0.2642	U. S. gallons
kilograms	2.205	pounds
cubic feet	7.480	U. S. gallons
imperial gallons	1.200	U. S. gallons

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